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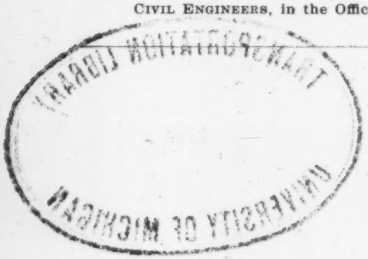

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ERRATA

Transactions, Vol. LXXXIII

Page 1398, line 13, for "-60×0.089", read "-55×0.0898".

Page 1398, line 15, for "$= 13.937 + 6.497 - 16.371 = 15.21$ ft." read "$= 25.076 + 6.497 - 16.371 = 15.202$ ft.".

Transactions, Vol. LXXXIV

Page 200, line 2, for "$\frac{y}{\delta} = \frac{M + Py_0}{M}$", read "$\frac{y_0}{\delta} = \frac{M + Py_0}{M}$".

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Paper No. 1463

GRAVITY AND ARCH ACTION IN CURVED DAMS.

BY FRED A. NOETZLI,* ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. F. J. NEUBAUER, ALLEN HAZEN, H. J. KESNER, A. F. PARKER, L. R. JORGENSEN, WILLIAM CAIN, B. F. JAKOBSEN, A. J. WILEY, CHAUNCY WERNECKE, EDWARD WEGMANN, D. C. HENNY, WILLIAM A. MILLER, L. LUIGGI, HUGO RITTER, AND FRED A. NOETZLI.

SYNOPSIS.

Dam engineering has reached a stage where it should be practiced as an exact science. The present method of designing arch dams by means of the cylinder formula is very unsatisfactory in many respects. On the other hand, all mathematical solutions of the problem of curved dams, published heretofore, are rather complicated and will hardly be used by the average engineer, as past experience has shown.

The writer shows in this paper some simple methods for determining in an arch dam the proportion of load carried either by gravity, that is to say, vertical cantilever action, or by horizontal arching. The accuracy of these methods compares very favorably with that obtained by the purely mathematical solution of the problem of arched dams, and, on the other hand, they are so simple that any engineer should be able to use them with ease.

By applying these methods to some typical examples of existing arch dams it is shown:

1.—That no curved dam of gravity section can be depended on to have any amount of water pressure supported by horizontal arching at the time of low temperature, unless the dam should have failed before in gravity action.
2.—That probably all existing curved dams of reduced gravity section have failed in gravity action in that they have cracked either between the masonry and the foundation, or in the masonry itself.
3.—That the usual method of calculating the stresses in arch dams by the well known "cylinder formula" fails to furnish correct values.

Further, some simple approximate formulas are developed for the determination of the temperature and shrinkage stresses in arch dams. It is shown that these secondary stresses in general are at least as high as the assumed maximum stresses from water pressure alone and that in every instance they are higher than would be tolerated for any other unreinforced masonry structure of scientific design.

* San Francisco, Cal.

Attention is also called to the fact that the factor of safety of a straight gravity dam is much lower than generally is assumed, and a method is given to calculate the soil pressure for a curved dam considered as a monolith of "hoof" form.

This paper was completed in manuscript when the attention of the writer was called to the excellent paper, "Arched Dams",* by B. A. Smith, M. Am. Soc. C. E. It seems to be a most fortunate coincidence that Mr. Smith, in a purely mathematical way, and the writer more with graphical methods, have investigated the problem of arched dams and have submitted their findings to the Engineering Profession at about the same time. Both consider the present method of calculating the stresses in arched dams by the cylinder formula as very unsatisfactory and absolutely erroneous, and both Mr. Smith's analytical and the writer's graphical methods lead (directly or indirectly) to conclusions which are so similar in many respects that either method may be regarded as checking the results of the other.

Mr. Smith's method requires a considerable knowledge of mathematics and, although his solution is exceedingly interesting from the theoretical point of view, the average engineer hardly will be able to use it to a great extent. Further, the solution given is applicable only to some special cases.

The writer's method of investigating arched dams is based, in the main, on the simple graphical construction of force and string polygons with which every engineer is familiar. Further, this method can be used for all kinds of arched dams without any limitations.

Introduction.

Ever since it was shown by the bold construction of the Zola Dam† in France and the old Bear Valley Dam‡ in California, that a curved dam is able to resist successfully the water pressure, even with a very much reduced gravity section, it has been the endeavor of all progressive engineers to take advantage of this fact, and to abandon, wherever possible, the old standard of a gravity dam in favor of the arched type. Unfortunately, these pioneers of dam construction had to be guided more by their engineering instinct than by facts supported by calculations. As a consequence, the curved type of storage dam, notwithstanding the numerous examples of most successful construction during the last two decades, still is looked upon by many as being only in the experimental stage.§

As a matter of fact, all the investigations published to date with regard to the statical conditions of curved dams have failed to show in a convincing manner the real possibilities of this kind of arch structure and to make available also in dam design the fine qualities of the arched form of masonry structure, of which the many advantages were recognized partly by the old Romans and are used to-day without question in bridge construction. On the other hand, apprehension

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 2027.

† M. de Tournadre in *Annales des Ponts et Chaussées*, 1872, and any good book on dam design.

‡ This dam, which was built in 1884, was superseded in 1910 by a higher structure a short distance farther down stream, so that the old dam now is submerged. A detailed description of the old dam, with an investigation of the static conditions, was published by H. Visher and L. Wagoner, M. Am. Soc. C. E., in *Transactions*, Technical Soc. of the Pacific Coast, 1889.

§ A. P. Davis, Past-President, Am. Soc. C. E., and D. C. Henny, M. Am. Soc. C. E., in *Transactions*, International Eng. Congress, 1915, Waterways and Irrigation, p. 694.

has been expressed that, in the case of arched dams, the curvature introduces unknown elements into the problem of design.

Owing to this state of uncertainty with regard to curved masonry dams, many engineers, and to some extent also the public, still favor the solid straight gravity type even in locations where an arch dam would be feasible. Such a gravity dam, which is assumed to depend for its stability entirely on its own weight and friction, is in general very uneconomical, as the real strength of the material under normal load conditions is used only to a moderate degree. Such a dam, notwithstanding the great quantity of material it requires, has only a comparatively small factor of safety, the factor against "overturning" generally being assumed as about 2. Further, in the case of high dams across narrow canyons, the general assumptions for the determination of the stresses in a pure gravity dam are not correct. The weight of the upper middle portion of the dam does not act vertically on the base, but is partly supported sideways by arching,* so that little is known about the real factor of safety of these structures.

It is easy to demonstrate that a dam which in plan is curved in an up-stream direction in such a way that a large part of the water pressure is transmitted laterally to the abutments, is much more economical with regard to the material required.

If it is known in what way cantilevers and arches act together under all conditions, it will be possible, by taking account of both actions, to design a curved dam with a cross-section smaller than either for a gravity or a pure arch dam.†

Unfortunately, an arch dam is many times statically indeterminate, due mostly to the restraint of the lower arch rings at the foundation. Modern arch design, based on the theory of the elastic arch, enables the determination with great accuracy of the stresses in any true arch; but in the case of a curved masonry dam there exists a combination of arching with vertical cantilever action. It is impossible that the lowest arch should deflect any appreciable amount under normal conditions, and only higher up, where the vertical cantilever has deflected elastically somewhat under the water pressure, arch action may develop and part or all of the water pressure will be transmitted laterally to the abutments. Therefore, the complicated case of a vertical cantilever beam which, besides having one end fixed, is supported along its entire length by elastic supports of varying stiffness formed by the horizontal arch rings, is encountered.

A great variety of opinion exists among authorities as to just how such a case should be treated, as no theoretically exact solution is possible. Some maintain that even curved dams in general should be designed with gravity section, regardless of the curvature. Others try to calculate such dams as arches only, and neglect entirely the gravity action.‡

* This fact has been pointed out by many, though hardly ever considered in practical designs. Compare D. C. Henny, M. Am. Soc. C. E., in *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 722.

† F. E. Turneaure and H. L. Russell, "Public Water Supplies", Second Edition, p. 389.

‡ Delocre, "Mémoire sur la Forme du Profil à adopter pour les Grands Barrages en Maçonnerie des Réservoirs," *Annales des Ponts et Chaussées*, 1866. (M. Delocre states that a curved dam will act as an arch if its thickness does not exceed one-third the radius of the up-stream side.)

Rankine, *The Engineer*, January, 1872.

Pelletreau, "Mémoire sur les Murs qui supportent une Poussée d'Eau", *Annales des Ponts et Chaussées*, 1876 and 1877. (M. Pelletreau fixes as a condition for the possibility of horizontal arching, that the thickness of the dam be not greater than one-half the up-stream radius.)

Krantz, "A Study on Reservoir Walls," New York, 1883. (According to Mr. Krantz a curved dam with a radius of 65 ft. or less will transfer the water pressure by horizontal arching on the hillsides, whatever the height of the structure.)

It is easy to prove that in the case of a long and comparatively low dam, the dam would have to fail first in its gravity action, before any perceptible arching could develop, as the upper arches in case of a lowering in temperature tend to deflect more than the maximum cantilever deflection under full load.

On the other hand, for a high dam with short radius and slender cross-section, the deflection of the vertical cantilever, due to the water pressure, might tend to be so great as to exceed considerably the deflection of the upper arch rings, so that these have to support the full water pressure, and, eventually, even more.

A number of excellent investigations have been published with regard to the curved dam problem, giving in an analytical way approximate solutions,* but the labor necessary for practical calculations is somewhat appalling to contemplate. This is probably the main reason why these theories very seldom have been applied in the design of new dams.† Most engineers prefer to use only the simple cylinder formula, and to cover the uncertainties of the design, as sometimes is frankly admitted, by a high factor of ignorance, that is, by assuming low unit stresses. Very seldom has any attention been given to the influence of a shortening of the arches due to shrinkage or to a lowering of the dam temperature, yet this action of the arches may be of much greater importance with regard to their percentage of water load, than the arch shortening due to water pressure itself. Besides, certain bending moments resulting from changes of the dam temperature, etc., will produce stresses which are often just as high or even higher than the stresses due to full water pressure.

As a matter of fact, it seems to be most unscientific that a structure like a dam, for which the load as a whole can be determined so accurately, and in which no vibration and, in general, no impact will occur, should be designed under the assumption of such low unit stresses as present practice favors. If the actual stresses in any part of a dam can be determined with a fair degree of accuracy, it will be possible to allow higher unit stresses and to build dams with a considerable saving in cost, and, at the same time, with a factor of safety at least as high as in any other engineering structure.

In what follows there will be developed a method by which it is possible to determine in a simple manner whether a curved dam has to be figured as a gravity section alone, or if arch action may be counted on, how much of the water pressure is supported by the vertical cantilever and how much by the horizontal arches.

Attention is also called to the fact that, for a dam which in plan is curved in an up-stream direction, the calculations with regard to gravity action cannot be made in the same manner as for a straight dam. The investigation of the

* Messrs. H. Visher and L. Wagoner, *Transactions*, Technical Soc. of the Pacific Coast, 1889; S. H. Woodard, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIII; the late R. Shirreffs, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIII, p. 155; Edwin Duryea, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIII, p. 172; B. A. Smith, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII, p. 2027.

† Favorable exceptions are the investigations made by the late G. Y. Wisner, M. Am. Soc. C. E., and E. T. Wheeler, M. Am. Soc. C. E., for the United States Reclamation Service, in connection with the plans for the Pathfinder Dam in Wyoming. *Engineering News*, August 10th, 1905, and the discussion on the Huacal Dam in Mexico, by L. J. Mensch, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXVIII (1910), p. 610.

stability of a curved dam (soil pressure, etc.) has to be made for the structure as a whole ("cylinder hoof") and not only for an imaginary isolated vertical dam slice, as is usually done. Only after the external stability of such a structure has been established satisfactorily, may the internal stresses from combined cantilever and horizontal arch action also be investigated.

It has been the writer's desire to avoid in this paper, as far as possible, all long calculations which, in any case, because the whole problem can be solved by trial only, would probably also involve unnecessary theoretical refinements. By deriving some simple approximate formulas, combined with graphical diagrams, the writer has provided means for an easy, but nevertheless fairly accurate method of calculating the stresses in arched dams. As far as it has been found to be desirable, the formulas are based on the exact theory of the hingeless elastic arch, with the same assumptions and limitations as are used in bridge design. The approximations and short cuts used, while simplifying the formulas, involve only a comparatively small percentage of error.

As stated previously, the calculation of the stresses in a curved dam is a statically indeterminate problem. A very close approximation may be obtained by comparing the deflection curves of the arch crowns, first, under the assumption that the arches have to support all the load, and, second, that the entire water pressure is sustained by vertical cantilever action alone. The load is then distributed on both cantilever and arches in such a way that the deflections of identical points in both systems are the same. In most cases the distribution of load has to be made by trial, as will be demonstrated later.

Deflection of Vertical Cantilever.

Neglect for the present all arch action, and assume that the vertical cantilever is strong enough to support alone the entire water pressure; also assume for this particular investigation that the cross-section of the dam is of the theoretical triangular form. The vertical cantilever shall be considered as consisting of a radial dam slice 1 ft. thick and of the same thickness up stream and down stream.

Fig. 1 (*a*) shows the simplified cross-section of the dam, Fig. 1 (*c*) the triangular water load, and Fig. 1 (*b*) the deflection curve.

The following notation will be used:

H = height of dam;
p = weight of unit volume of water;
P_b = pressure per square unit at base of dam;
t_b = thickness of dam at the bottom;
x = elevation of horizontal section above base;
M_x = bending moment at Elevation x;
I_x = moment of inertia of cantilever at Elevation x,
I_b = moment of inertia of cantilever at base;
ρ = radius of curvature of deflection curve at Elevation x;
y = deflection at Elevation x; and
E = modulus of elasticity.

Under the pressure of the water the cantilever is deflected elastically in a down-stream direction. The differential equation of the deflection curve of the cantilever beam may be written in the well known form:

$$\frac{d^2 y}{d x^2} = \frac{M_x}{E I_x} = \frac{1}{\rho} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The bending moment at Elevation x above the base, due to the water pressure, has the value,

$$M_x = p \frac{H - x}{2} (H - x) \frac{H - x}{3} = p \frac{(H - x)^3}{6}.$$

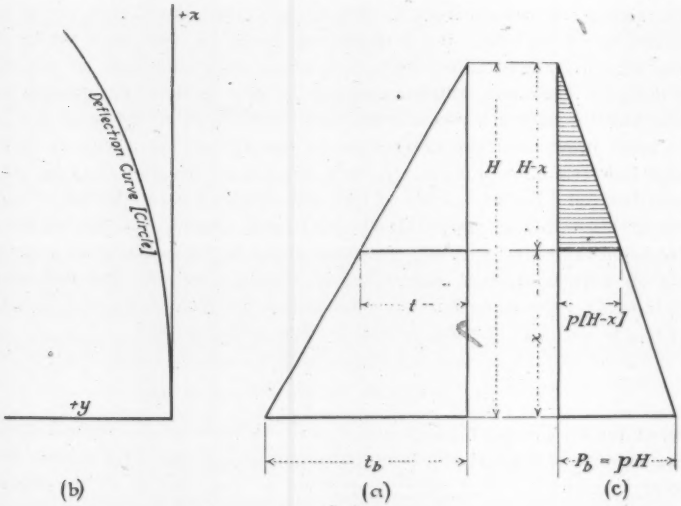

FIG. 1.

The moment of inertia of the cantilever is variable; at Elevation x above the base it has the value,

$$I_x = \frac{1}{12} t^3 = \frac{1}{12} \left(\frac{t_b}{H} \right)^3 (H - x)^3$$

Introducing these values in Equation (1), we obtain,

$$\frac{d^2 y}{d x^2} = \frac{2 p H^3}{E t_b^3} = \frac{1}{\rho} = \text{constant} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Equation (2) indicates that the radius of curvature, ρ, of the deflection line is constant for any elevation, x, or, in other words, we have the very interesting fact that the deflection line is with close approximation a portion of a circle. The radius of this circle is,

$$\rho = \frac{E t_b^3}{2 p H^3}.$$

By integration of Equation (2), we obtain,

$$\frac{d\,y}{d\,x} = \frac{2\,p\,H^3}{E\,t_b^3}\,x\,(+\,C_1 = 0)$$

and,

$$y = \frac{p\,H^3}{E\,t_b^3}\,x^2\,(+\,C_2 = 0) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

By means of Equation (3), we are able to figure the deflection, y, of the cantilever at any elevation, x, above the base. For the construction of the entire deflection line, we can make use of the fact that it is a circle, for which a vertical line at the fixed end is a tangent. This circle also has to go through the point of maximum deflection at the top of the dam, where

$$y_{max.} = \frac{p\,H^5}{E\,t_b^3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4a)$$

Equation (4a) may also be written in the form,

$$y_{max.} = \frac{p\,H^5}{12\,E\,I_b} = \frac{P_b\,H^4}{12\,E\,I_b} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4b)$$

For a cantilever of a constant moment of inertia, I_b, we have for the maximum deflection the well known value,

$$y_{max.} = \frac{p\,H^5}{30\,E\,I_b} = \frac{P_b\,H^4}{30\,E\,I_b} = \frac{p\,H^5}{2.5\,E\,t_b^3} \ldots\ldots\ldots\ldots\ldots\ldots (4c)$$

Comparing Equation (4a) with Equation (4c), we see that, for triangular load, a cantilever, as shown in Fig. 1 (a), deflects at the free end exactly two and one-half times as much as a cantilever of constant moment of inertia, I_b. Use of this fact will be made later.

Deflection of the Horizontal Arch Slices.

In general, the curvature in arch dams is that of true circles. For the statical calculations of such dams it will be found very convenient if the length of a circular arc can be calculated from the length of the chord and the rise of the arc, which dimensions are always known for every horizontal arch slice of a dam. A simple approximate formula will be developed, as follows:

Let L = length of circular arc (See Fig. 2);
l = chord;
h = rise;
R = radius; and
ϕ = central angle.

Between these values the following relations exist:

$$L = R\,\phi \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

$$l = 2\,R \sin \frac{\phi}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

$$h = R - R \cos \frac{\phi}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Adding the square of Equation (6) to four times the square of Equation (7), we obtain

$$l^2 + 4\,h^2 = 4\,R^2\left(\sin^2\left(\frac{\phi}{2}\right) + \cos^2\left(\frac{\phi}{2}\right)\right) - 8\,R^2\cos\frac{\phi}{2} + 4\,R^2$$

$$= 8\,R^2\left(1 - \cos\frac{\phi}{2}\right)$$

By extending the value, $\cos\frac{\phi}{2}$, into a series and breaking off after the third term, we obtain

$$l^2 + 4\,h^2 = 8\,R^2\left\{1 - \left(1 - \frac{\left(\frac{\phi}{2}\right)^2}{2} + \frac{\left(\frac{\phi}{2}\right)^4}{2\times 3\times 4} - \ldots\right)\right\}$$

$$= R^2\,\phi^2 - \frac{R^2\,\phi^4}{48} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

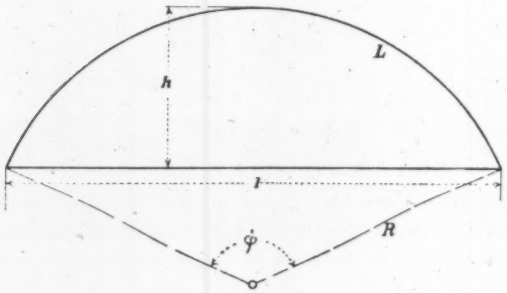

FIG. 2.

If in Equation (7) we also extend $\cos\frac{\phi}{2}$ into a series and break off this time after the second term, we have

$$h = R - R\left(1 - \frac{\left(\frac{\phi}{2}\right)^2}{2} + \ldots\right)$$

$$= R - R + R\,\frac{\phi^2}{8}$$

or,

$$h = R\,\frac{\phi^2}{8}$$

and,

$$h^2 = R^2\,\frac{\phi^4}{64} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

This value, multiplied with $\frac{4}{3}$, introduced in Equation (8) gives,

$$l^2 + 4\,h^2 = R^2\,\phi^2 - \frac{4}{3}\,h^2.$$

Under consideration of Equation (5), we obtain

$$L^2 = R^2 \phi^2 = l^2 + \frac{16}{3} h^2$$

and,

$$L = \sqrt{l^2 + \frac{16}{3} h^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (10a)$$

also,

$$h = \sqrt{\frac{3}{16}(L^2 - l^2)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (10b)$$

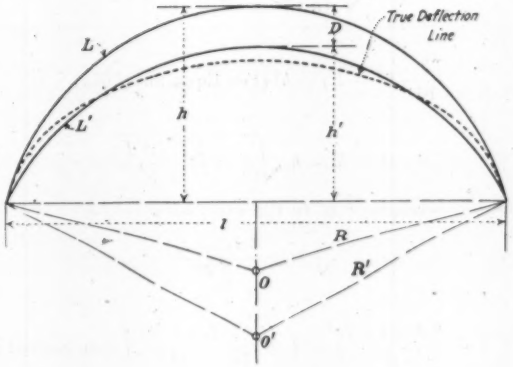

FIG. 3.

By means of Equation (10*a*), we are able to figure with great accuracy the length of an arc, if the length, *l*, of the chord and the rise, *h*, of the arc are known. The error involved is only a fraction of 1% for an arc with a central angle of 120° or less.

With the aid of Equation (10*a*), it will be easy to figure approximately also the deflection of arches. Under the influence of the axial forces in an arch, which result from the pressure of the water in the reservoir, or are due to any other reason, such as shrinkage, temperature, etc., an arch of the original length, *L*, is shortened somewhat and comes approximately into the position of the line, *L'*, in Fig. 3.

By using the same notations as before and calling,

ΔL = the shortening of the original length;

L' = the length of shortened arch;

h' = rise of shortened arch;

$D = h - h'$ = deflection of arch;

we have, under the assumption that the abutments are unyielding (Span l not changed) and that the ends of the arch may turn freely:

$$D = h - h'$$

$$= h - \sqrt{\frac{3}{16}\left\{(L - \Delta L)^2 - l^2\right\}}$$

$$= h - \sqrt{\frac{3}{16}\left\{[L^2 - 2L(\Delta L) + (\Delta L)^2] - l^2\right\}}$$

The value, $(\Delta L)^2$, is very small compared with the other factors in the root and may be neglected.

$$D = h - \sqrt{\frac{3}{16}\left\{L^2 - 2L(\Delta L) - l^2\right\}}$$

$$= h - \sqrt{\frac{3}{16}(L^2 - l^2) - \frac{3}{8}L(\Delta L)}$$

The value,

$$\frac{3}{16}(L^2 - l^2) = h^2 \text{ (See Equation (10}b\text{))}.$$

Therefore,

$$D = h - h\sqrt{1 - \frac{3L(\Delta L)}{8h^2}}$$

By applying the binomial theorem to the expression

$$\sqrt{1 - \frac{3L(\Delta L)}{8h^2}}$$

we obtain,

$$\left(1 - \frac{3L(\Delta L)}{8h^2}\right)^{\frac{1}{2}} = 1 - \frac{3}{16}\frac{L(\Delta L)}{h^2}\left(+ \text{neglected}\right)$$

and,

$$D = h - h\left(1 - \frac{3}{16}\frac{L(\Delta L)}{h^2}\right)$$

$$D = \frac{3}{16}\frac{L}{h}\Delta L \text{ (approximately)} \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

The arch deflections as obtained by Equation (11) are somewhat smaller than those which would be obtained by consideration of fixed abutments (see Fig. 3). The influence of shear on the deflection, in general, may be neglected as it is small in the case of arches.

Equation (11) also may be obtained by using Huyghens' approximation for the length, L, of a circular arc*:

$$L = \frac{8c - l}{3}$$

where l is the chord of the whole arc and c the chord of half the arc. If h is the rise of the arc, we have,

$$c = \sqrt{\frac{l^2}{4} + h^2}$$

* See "Mechanical Engineers' Pocket Book", by W. Kent, 9th Edition, p. 58.

and

$$L = \frac{8\sqrt{\frac{l^2}{4} + h^2} - l}{3}$$

By calculating again the deflection, D, as the difference of the rises, h and h', of the original and the shortened arch, respectively, we obtain in a similar way as previously shown, the equation,

$$D = h - h' = \frac{1}{64}\,\frac{9\,L + 3\,l}{h}\,\Delta\,L$$

For arches with a central angle of 120° or less, the value $(9\,L + 3\,l)$ differs not very much from the value $12\,L$.

Thus, we obtain again as an approximation,

$$D = \frac{3}{16}\,\frac{L}{h}\,\Delta\,L \qquad (11)$$

Equation (11) shows that the deflection, D, of the crown of an arch is directly proportional to the length, L, and the total shortening, $\Delta\,L$, of the arch center line, and inversely proportional to the rise, h. For flat arches the length, L, does not differ much from the span, l, and hence, approximately,

$$D = \frac{3}{16}\,\frac{l}{h}\,\Delta\,L$$

This formula is very convenient also in bridge design.

The shortening, $\Delta\,L$, of an arch under axial pressure is calculated by means of the relation,

$$\Delta\,L = \frac{\text{Unit Stress}}{\text{Modulus of Elasticity}} \times \text{Length} = \frac{f_c}{E}\,L.$$

Equation (11) then may be written in the form,

$$D = \frac{3}{16}\,\frac{f_c}{E}\,\frac{L^2}{h} \qquad (12)$$

The average stress, f_c, in any horizontal arch slice of a curved dam is determined approximately by dividing the axial thrust by the area.

In circular arcs with central angles between 40° and 110°, which, in general, are the practical limits in arched dams, the value $\frac{L^2}{h}$ varies only from 8.10 R, for an angle of 40°, to 8.64 R for one of 110° and, therefore, may be assumed as nearly constant.

This indicates that in the common type of arched dam, for which the radius of the center line of the horizontal arch slices is more or less the same, the deflections, D, at the crowns are approximately the same for the lowest arches as for the highest ones, irrespective of the length, provided the unit stresses in all arches are the same.

A limitation to this conclusion of course lies in the fact that, in the lower parts of a dam, the length of the arches becomes so small, as compared with their thickness, that it is out of the question to speak of arch action under such conditions. In those cases the "arches" act more like wedges, and real arch action occurs only when the thickness of the arch is small as compared with its length and radius.

The value, $\frac{L^2}{h}$, equals very nearly 8.3 R for most arched dams, and we may write Equation (12) in the simple form,

$$D = \frac{1.56}{E} R f_c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

or, because $f_c = \frac{P R_u}{t}$,

$$D = \frac{1.56}{E} \frac{P R R_u}{t} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13a)$$

where P is the water pressure per square unit, R_u, the up-stream radius, and t the dam thickness at the depth investigated. Equation (13a) in form compares favorably with the one derived in an entirely different way by the late Mr. R. Shirreffs,* but furnishes somewhat higher values for the arch deflections.

Equation (11) gives the deflection of an arch due to any reason for a shortening ΔL. A variation in temperature of $\pm T°$ produces a change in the length, L, of the arch of $\Delta L = c\ T\ L$, where c is the coefficient of linear expansion. By Equation (11) we obtain

$$D_t = \frac{3}{16} \frac{L^2}{h} c\ T$$

and by replacing $\frac{L^2}{h}$ by 8.3 R, we have,

$$D_t = 1.56\ R\ c\ T \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13b)$$

For the calculation of a deflection due to the shrinkage of the concrete, we may introduce the total shrinkage, ΔL, for the entire length of the arch directly into Equation (11), or consider this shrinkage as corresponding to a decrease in temperature.

Equation (11) also indicates that for an arch for which the quotient, $\frac{L}{h}$, equals $\frac{16}{3} = 5.33$ (central angle, 90° approximately), D equals ΔL, or, in other words, for arches with central angles of about 90°, the deflection, D, at the crown is very nearly equal to the shortening, ΔL, of the length of the arch. It may be stated again that the shortening or lengthening of the arches may be due to any cause, compression, shrinkage, variation in temperature, etc.

Combined Cantilever and Arch Action.

Every arched dam will be stressed, primarily, according to the laws of gravity action. This is due to the restraint at the foundation of the lowest arch, where any arch deflection is impossible if the bond between foundation and masonry is not destroyed.

Several attempts have been made by different authors to determine mathematically the proportions of load supported by the vertical cantilever and by the horizontal arches. Hardly anybody has ever taken into consideration the fact that the shortening of the arches, due to shrinkage of the concrete, a lowering in temperature, etc., may be much greater than that due to the direct axial stress. The maximum deflec-

* *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 89.

tion of the horizontal arches, therefore, will be much greater than is generally assumed. The following example is a typical illustration.

Assume in a curved dam a horizontal arch slice which has a length, $L = 100$ ft. and a rise, $h = 15$ ft. The shrinkage of the concrete may cause a shortening of the length of the arch of about $\Delta L = \frac{1}{4}$ in., or 0.021 ft. If the temperature falls 20° below the closing temperature of the arch, a shortening,

$$\Delta L_t = 0.000\,0055 \times 20 \times 100 = 0.011 \text{ ft.}$$

will result, if c is assumed to be 0.000 0055. The shortening due to water pressure, which may produce a stress of $f_c = 300$ lb. per sq. in., would be, with $E = 2\,000\,000$ lb. per sq. in.,

$$\Delta L_f = \frac{f_c}{E} L = \frac{300}{2\,000\,000} 100 = 0.015 \text{ ft.}$$

The total shortening of the original length, L, is

$$\Delta L = \Delta L_s + \Delta L_t + \Delta L_f = 0.021 + 0.011 + 0.015 = 0.047 \text{ ft.}$$

This corresponds to a deflection of the arch at the crown of,

$$D = \frac{3}{16} \frac{L}{h} \Delta L = \frac{3}{16} \times \frac{100}{15} \times 0.047 = 0.059 \text{ ft.} = 0.76 \text{ in.}$$

The deflection due to water pressure alone would be

$$D_w = \frac{3}{16} \frac{L}{h} \Delta L_f = \frac{3}{16} \times \frac{100}{15} \times 0.015 = 0.019 \text{ ft.} = 0.23 \text{ in.}$$

The total arch deflection due to shrinkage, temperature, and water pressure, therefore, would be more than three times as great as the deflection due to water pressure alone.

The easiest way to solve the problem of combined cantilever and arch action is by means of the deflection curves, and consists in distributing by trial the water load on both cantilever and arches in such a manner that the deflections of identical points, calculated separately for cantilever and arch system under partial load and with due consideration of shrinkage and temperature deformations, are identical. Ice pressure, whenever it may occur, has to be treated in a manner similar to water pressure.

As a basis for such an investigation, assume that a dam may be imagined as consisting at one time of separate vertical slices of constant thickness, considered as vertical cantilevers, and at another time of horizontal arch slices. Both vertical and horizontal dam slices are preferably taken 1 ft. thick and shall be considered as free to move relatively to each other.

With regard to the load on the cantilever, it is known that at the fixed end at the foundation all the water pressure has to be supported by cantilever action. Nothing else is known *a priori* about the load conditions higher up.

There are two different cases possible:

Case 1.—The cantilever is so strong that its deflection at the top, under the assumption that the cantilever takes all the load, is less than the deflection of the upper arch rings if these would have to sustain the entire water pressure (considering also the effect of shrinkage and temperature). In such a case (Fig. 4) practically all the load is supported by cantilever action, and horizontal arching

may occur only when the arch is lengthened by a rise in temperature. Such conditions exist, for example, in a curved dam of gravity section.

Case 2.—A comparatively long and high dam of rather slender cross-section, for which apparently considerable arch action must exist to insure its stability.

The cantilever, due to its rigidity near the foundation, will have to support practically all the water load near the bottom until its deflection in a down-stream direction is so great that appreciable horizontal arch action may develop and help in sustaining the water pressure. In the upper parts of the dam, the deflection of the cantilever may be so great that part or all the water pressure is supported by horizontal arching.

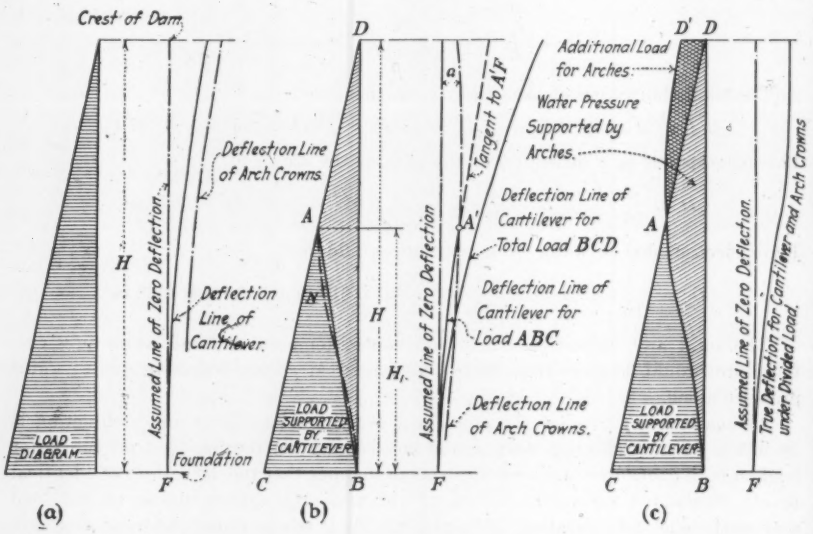

FIG. 4.

APPROXIMATE DETERMINATION OF WATER PRESSURE CARRIED BY THE VERTICAL CANTILEVER.

The correct analytical investigation of Case 2 is very difficult, if not impossible, especially on account of the statically uncertain conditions of the lower arches. As to actual load on the cantilever, neglecting the upper elastic supports, it is known only that immediately above the foundation it amounts to the full water pressure and diminishes to zero at or near a point, A', Fig. 4 (*b*), where the deflection curve of the partly loaded cantilever intersects the deflection line of the arches under full load. Above A' the horizontal arches have to support the entire water pressure.

An investigation shows that the true load of the cantilever decreases at nearly uniform rate from the maximum at the bottom up to the point where the cantilever would deflect about as much as the arches at the same elevation and under full load (considering also shrinkage and temperature). The determination of the exact cantilever load area, *B C A N*, Fig. 4 (*b*), however, is rather complicated. As an

approximation, assume that the cantilever is supporting the triangular load $B\,C\,A$, instead of the real load $B\,C\,A\,N$. The difference between the two loads, in general, is small. The horizontal arches, then, have to be calculated as carrying the rest of the load, triangle $B\,A\,D$.

For the approximate determination of the size of the cantilever load, triangle $B\,C\,A$, Equation (4*b*) or Equation (4*c*) may be used in connection with the assumption already stated, that direct cantilever action ceases at or near the elevation, H_1, Fig. 4 (*b*), where the deflection of the partly loaded cantilever is equal to the deflection of the horizontal arch at the same elevation and under full water pressure (considering also the influence of shrinkage and temperature). The error involved is on the side of safety, as far as the cantilever is concerned. The value of the deflections a of the arches at any elevation may be computed by Equations (11), (13*a*), and (13*b*).

After having determined the arch crown deflection curve (three or four points in general are sufficient to draw the curve), compute by trial, that is, by repeated application of Equations (4*b*) and (4*c*) for triangular load, for which elevation, H_1, the cantilever loaded by a certain triangle $B\,C\,A$ deflects at point A approximately as much as the horizontal arch at the same elevation. It has to be considered that in these equations the factor P_b (base of the load triangle) equals the water pressure per square unit at the bottom of the dam with the reservoir full, but for H the assumed height H_1 of the point A above the base is introduced. Thus,

$$y = \frac{P_b\,H_1^4}{E\,t_b}$$

if the thickness of the vertical cantilever under consideration changes from t_b to zero between the base and elevation, H_1. If the dam thickness is constant or nearly so, then the deflection is,

$$y' \doteq \frac{P_b\,H_1^4}{2.5\,E\,t_b^3}$$

For the actual case of a trapezoidal cantilever, an interpolation has to be made between the values of y and y'.

If H_1 as finally obtained in this way should be greater than the height H of the dam, it would indicate that under the assumed conditions the arches, due to shrinkage and low temperature, have the tendency to deflect more than the cantilever under full water pressure so that the latter would not be supported at all by any arch. It can be shown that this is the case with all curved gravity dams, so that at times of low temperature these dams would have to fail in gravity first before arch action could develop. At a time of high temperature, of course, considerable arching may exist, but in computing the stability of a dam it is not proper, for obvious reasons, to take advantage of this fact.

Also, it is possible to determine in this very simple way and with a fair degree of accuracy, the division of the water load between cantilever and arches for any dam of reduced gravity section. It will then be easy to decide whether an assumed profile under the given circumstances will act mainly by gravity or by arching. The nearer the point A is to the top of the dam, the greater will be the area of the cantilever load triangle $B\,C\,A$ as compared with the total water pressure triangle

B C D and the greater is the percentage of the load supported by the cantilever. If the division of the total load is determined in this way, the investigation of the stresses in both cantilever and arches may then be made as usual and with due consideration also of the vertical load for the cantilever (weight of the dam, etc.).

This reasoning holds good only so long as the cantilever really is strong enough to support its entire share of the load. There is, however, the possibility that with a reduced cross-section of the dam the stresses in the cantilever are too high, so that it will fail in its gravity action and produce cracks. In this case a much higher percentage of the load is thrown upon the arches.

Graphical Determination of Division of Load Between Cantilever and Arches.

For important structures a greater degree of accuracy will be desirable than is obtained by applying the method previously outlined.

It is possible to obtain a fairly good approximation by calculations. Such methods were developed as early as 1886 by H. Visher and B. Wagoner,* M. Am. Soc. C. E., in an investigation of the old Bear Valley Dam, and in 1904 by S. H. Woodard,† M. Am. Soc. C. E., in the design of the Lake Cheesman Dam in Colorado. The amount of work required, however, is rather extraordinary, especially when the deformations due to shrinkage and temperature are considered. Apparently, little use of these theories has been made thus far.

A much easier method, which requires in general only a small percentage of the time necessary for an analytical investigation, is a graphical one. Just as in graphic statics certain problems are solved graphically much more quickly and clearly than by cumbersome calculations, so a graphical construction is adapted admirably to the problem of curved dams. The results of such an investigation will be the more accurate the thinner a dam is compared to its height.

It is a well known principle in statics that the elastic line of a beam can be found graphically as the funicular polygon for the beam loaded by its moment area.‡ In the case of a beam with varying moment of inertia, *I*, the ordinates of the moment area have to be divided in each section of the beam by the respective *I*. The area thus obtained and applied as load area to the beam furnishes (in the funicular polygon to proper scale) the deflection line.

The application of these principles to the problem of the curved dam, may be sketched roughly as follows:

As a first assumption, determine the approximate load triangle *B C A* in the same manner as described on page 14. Then construct the cantilever deflection line due to this load, and compute separately the arch crown deflection curve due to the remaining load and under consideration of the possible extremes of shrinkage and temperature deformations. As these two lines in general will not be coincident, the load distribution upon cantilever and arches is corrected and

* *Transactions*, Technical Soc. of the Pacific Coast, 1889.

† *Transactions*, Am. Soc. C. E., Vol. LIII. Another analytical solution of the problem of arched dams was given by Mr. B. A. Smith in *Transactions*, Am. Soc. C. E., Vol. LXXXIII, p. 2027.

‡ W. Ritter, "Anwendungen der Graphischen Statik", Vol. 3, Zurich, 1900. *See also* "Mechanical Engineers' Handbook", by L. S. Marks, First edition, 1916, p. 412.

the construction repeated, until the two deflection curves for cantilever and arch crowns are as nearly identical as practicable.

It will often be found that the cantilever, due to its stiffness at the bottom and the consequent large load (area $B\ C\ A$), tends to deflect at the free end, even without any load above A, more than the corresponding arches under full water pressure, etc. These arches therefore have to support the free end of the cantilever to prevent it from deflecting more than they do themselves. The additional load upon the upper arches is represented by the area $A\ D\ D'$, Fig. 4 (*c*).

It will be seen that in the case of high dams of slender cross-section, the load $A\ D\ D'$ (which acts upon the cantilever in opposite direction to the load $B\ C\ A$) may be quite considerable. As this load $A\ D\ D'$ represents the additional amount of pressure acting upon the arches due to the reaction of the upper part of the cantilever, it cannot be neglected in the design of important structures. In such a case certain upper arches, besides the full water pressure, have to be calculated as carrying additional load in supporting the free end of the cantilever. Also the bending in the cantilever near A' may have to be considered.

Example—Pathfinder Dam.

To illustrate the method of the graphical determination of the division of the water pressure between vertical cantilever and horizontal arches, the Pathfinder Dam will be investigated. This structure, one of the highest curved dams of reduced gravity section, was built in 1905-07 by the United States Reclamation Service. The fact that similar investigations* have been made for this dam makes this example particularly suitable for the writer's purpose, because in this way the results of the analytical and the graphical methods may be compared.

The Pathfinder Dam has a maximum height of $H = 210$ ft. and a thickness of $t_b = 94$ ft. at the base and $t = 10$ ft. at the crest. The cross-section of the dam is nearly symmetrical, the batter of the up-stream face being 0.15 ft. and that of the down-stream face 0.25 ft. The radius at the top is 150 ft., varying but slightly in the lower portions; for calculations, therefore, consider the radius as constant, $R = 150$ ft. The dam is built of cyclopean masonry for which a modulus of elasticity of $E = 3\,000\,000$ lb. per sq. in. will be assumed.

For the present investigation, consider a vertical dam slice of 1 ft. uniform thickness, located at the center of the dam, and elementary arches consisting of horizontal dam slices of 1 ft. thickness, each assumed to be free to move relatively to the other. Assume the water to be at a level with the crest of the dam and that the vertical dam slice, considered as an independent cantilever, carries the water pressure without any support from horizontal arching. If the cantilever was of true triangular shape, the elastical deflection line would be a circular arc, and the maximum deflection at the top, according to Equation (4*a*), would be,

$$y = \frac{p\,H^5}{E\,t_b^3} = \frac{62.5 \times 210^5}{3\,000\,000 \times 144 \times 94^3} = 0.071 \text{ ft.} = 0.85 \text{ in.}$$

* By G. Y. Wisner and E. T. Wheeler, *Engineering News*, August 10th, 1905.

If the cantilever was of constant moment of inertia, Equation (4*c*) would give for the maximum deflection the value,

$$y' = \frac{p\,H^5}{2.5\,E\,t_b^{\,3}} = \frac{y}{2.5} = 0.028 \text{ ft.} = 0.34 \text{ in.}$$

The true deflection of the theoretical cantilever will be somewhere between the values of y and y' and probably much nearer y, as the cantilever is more triangular than rectangular in shape.

Fig. 5 shows how the deflection line of the cantilever of varying moment of inertia and loaded by the triangular water load can be found graphically. First compute for sections at different elevations the moment of inertia and the bending moment due to the water pressure. Column (1) of Table 1 gives the elevations of the horizontal sections through the cantilever, Column (2) shows the corresponding

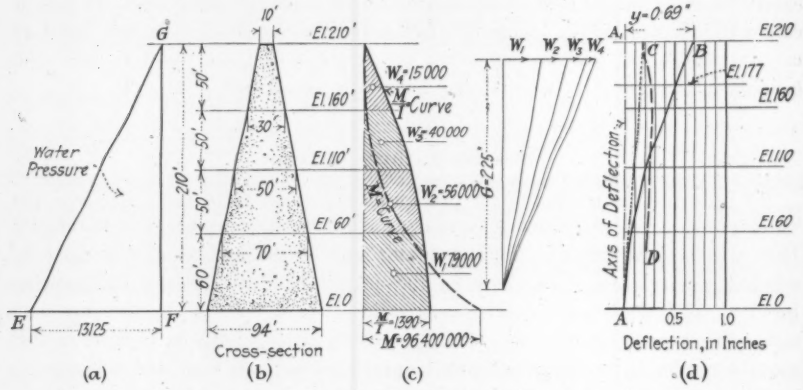

FIG. 5.

moments of inertia, I, and Column (3) gives the bending moments M due to the portion of the load above the elevation under consideration, calculated from the formula

$$M = \frac{p\,H_x^{\,3}}{6}$$

if H_x is the distance of the section below the crest of the dam. In Column (4) are given the values $\frac{M}{I}$. All dimensions are in feet and the weights in pounds.

TABLE 1.

(1) Elevation.	(2) Moment of Inertia, I	(3) Bending Moment, M	(4) $\frac{M}{I}$	(5) Load Area, W
210	830	0	0	15 000
160	2 250	1 300 000	580	40 000
110	10 400	10 400 000	1 000	56 000
60	28 600	35 100 000	1 230	79 000
0	69 300	96 400 000	1 390	

Fig. 5 (*b*) shows a cross-section of the dam and Fig. 5 (*a*) the triangular load due to water pressure. In Fig. 5 (*c*) are plotted the values of M and $\frac{M}{I}$, whereby the M and $\frac{M}{I}$ curves are obtained, and from which are taken the load areas W to use in Fig. 5 (*d*), where the force and string polygons have been drawn in the ordinary way. The pole distance G of the force polygon is preferably taken such that the deflections in any elevation of the dam, measured upon the drawing, are obtained in the actual size as they occur at the dam. The general theory of the graphical determination of deflection curves shows that this is the case when the pole distance is

$$G \text{ ft.} = \frac{E \text{ lb. per sq. ft.}}{\text{Scale of Load Area, } W, \times \text{ Scale of Dam}}$$

For this case,

$$E = 432\,000\,000 \text{ lb. per sq. ft.}$$

Scale of load areas W:

$$1 \text{ ft.} = 2\,400\,000 \text{ units of } W, \text{ or } 1 \text{ in.} = 200\,000 \text{ units of } W.$$

Scale of dam:

$$1 \text{ ft.} = 960 \text{ ft., or } 1 \text{ in.} = 80 \text{ ft.}$$

Therefore

$$G = \frac{432\,000\,000}{2\,400\,000 \times 960} = 0.187 \text{ ft.} = 2.25 \text{ in.}$$

With this pole distance, $G = 2.25$ in., the force and string polygons have been drawn. The curve inclosed by the string polygon is the elastic line of the theoretical cantilever (Curve $A\ B$ of Fig. 5 (*d*)). The maximum deflection as obtained in this way is 0.69 in. and this checks well with the results from Equations (4*a*) and (4*c*), from which an interpolation would have led to about the same value.

To obtain the theoretical arch crown deflection line, assume that all the water pressure is supported by horizontal arching and that the arches are shortened by a decrease of temperature. Assume that the drop of the arch temperature below normal is 15° Fahr.* at the crest of the dam, diminishing at a uniform rate to nil at the bottom. The corresponding arch deflection at the crest, with the linear coefficient of expansion $c = 0.000\,005$, is by Equation (13*b*),

$$D_t = 1.56\ R\ c\ T = 1.56 \times 150 \times 0.000\,005 \times 15 = 0.0175 \text{ ft.} = 0.21 \text{ in.}$$

The arch deflections at the lower elevations decrease proportionally to the height, as shown in Column (2) of Table 2.

The theoretical arch deflections due to the water pressure are obtained from Equation (13*a*),

$$D_w = \frac{156}{E} \frac{P\ R\ R_u}{t}$$

These deflections are given in Column (3) of Table 2.

The theoretical arch deflections are shown in Fig. 5 (*d*), where the dotted line $A\ C$ is the deflection of the arch crowns due to a decrease of temperature alone

* It is assumed for this example that a decrease in temperature of 15° Fahr. occurs only because Mr. Wheeler also used this value. In this way the results of the two investigations may be compared more easily. But the maximum drop of temperature at the crest of the Pathfinder Dam is probably considerably more than 15° Fahr.

TABLE 2.

(1) Elevation.	(2) D_t, in feet.	(3) D_w, in feet.	(4) Arch Deflections: Feet.	(5) Inches.
210	0.0175	0.0	0.0175	0.21
160	0.0133	0.0093	0.0226	0.27
110	0.0092	0.0119	0.0211	0.25
60	0.0050	0.0134	0.0184	0.22
0	0.0	0.0149	0.0149	0.18

and the dash line $C\,D$ is the deflection line due to temperature and full water pressure.

The water pressure will be divided between cantilever and arches in such a way that the resulting deflection lines are identical in both systems. It is evident that the real deflection line in the upper portion will terminate somewhere between the points B and C.

With rough approximation, the amount of water pressure carried by the cantilever may be obtained by using the method outlined on page 14. By repeated application of Equations (4*b*) and (4*c*) it will be found that a load triangle $E\,F\,K$, Fig. 6 (*a*), with the corner K at an elevation of about 177 ft. above the base, deflects the cantilever at this elevation for about as much as the elementary arch at the same elevation is deflected due to the full water pressure and a decrease in temperature.

The easiest way to get this result is to judge approximately by inspection of Fig. 5 (*d*), at which point the deflection curve of the partly loaded cantilever will intersect the arch deflection curve $C\,D$. Assume a deflection $y = 0.022$ ft. (0.26 in.) for this point. Equation (4*a*) for a triangular cantilever gives for H_1 the value

$$H_1' = \sqrt[4]{\frac{y\,E\,t_b^3}{P_b}} = \sqrt[4]{\frac{0.022 \times 3\,000\,000 \times 144 \times 94^3}{13\,125}} = 157 \text{ ft.}$$

and Equation (4*c*) for a rectangular cantilever gives

$$H_1' = H_1' \sqrt[4]{2.5} = 197 \text{ ft.}$$

A rough interpolation gives

$$H = 177 \text{ ft.}$$

To check this calculation by the more accurate graphical construction, the values in Table 3 were obtained in a similar way to those in Table 1. The cantilever load is now represented by the triangle $E\,F\,K$, Fig. 6 (*a*), which has the same base $E\,F$ but less height than the full load triangle $E\,F\,G$.

Fig. 6 (*c*) shows the M and $\frac{M}{I}$ curves, and in Fig. 6 (*d*) the force and string polygons are drawn for the cantilever loaded by the $\frac{M}{I}$ areas, W. It is seen that the elastic curve $A\,B_1$ of the cantilever loaded by the triangle $E\,F\,K$, intersects the arch deflection curve $C\,D$ at about Elevation 172. This checks fairly well with the value, 177 ft., obtained by the approximate analytical method.

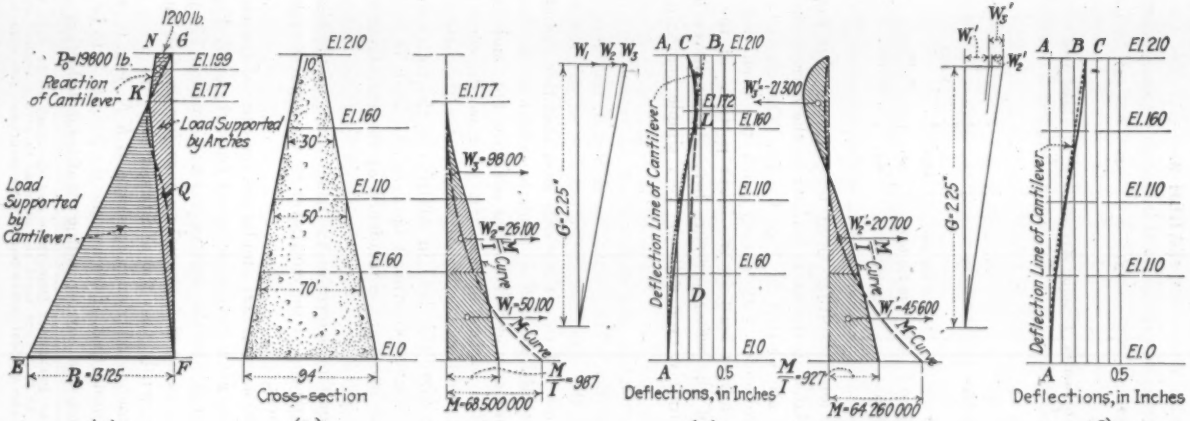
1200 lb.
N
G
P1=19800 lb.
El. 189
Reaction of Cantilever
K
El. 177
Load Supported by Arches
Load Supported by Cantilever
Q
E
P2=13125
F
(a)
El. 210
10'
El. 160
30'
El. 110
50'
El. 60
70'
El. 0
94'
Cross-section
(b)
El. 177
W1 W2 W3
W3=9800
W2=26100
M/I Curve
W1=50100
M/I Curve
G=225"
M/I=987
M=68500000
(c)
A, C B, El. 210
Deflection Line of Cantilever
El. 172
L
El. 160
El. 110
D
El. 60
El. 0
A
0.5
Deflections, in Inches
(d)
W2'=21300
W3'=20700
M/I Curve
W1'=45600
M/I Curve
M/I=927
M=64260000
(e)
W3'
W1'
W2'
G=225"
A B C El. 210
El. 160
El. 110
El. 110
El. 0
Deflection Line of Cantilever
A
0.5
Deflections, in Inches
(f)

FIG. 6.

TABLE 3.

(1) Elevation.	(2) Moment of Inertia, I	(3) Bending Moment, M	(4) $\frac{M}{I}$	(5) Load Area, W	(6) (7) Arch Deflections, Temperature and Partial Load:	
					Feet.	Inches.
210	83	0	0	0	0.0175	0.21
177	1 040	0	0			
160	.....	.....	...	9 800	0.0188	0.23
110	10 400	3 700 000	356		0.0118	0.14
60	28 600	19 700 000	689	26 100	0.0061	0.07
0	69 300	68 500 000	987	50 100	0	0

The arch deflections resulting from temperature and partial load (triangle $F\ G\ K$) were calculated in the same manner as the corresponding values in Table 2. The deflections are plotted in Fig. 6 (d). The elastic curve of the partly loaded cantilever coincides very closely with the arch deflection line for the remainder of the load, at least in the lower portions of the dam. This proves that the division of the water pressure between cantilever and arches may be obtained fairly accurately, even with the very simple approximate method as outlined on page 14.

According to assumption, the arches above Elevation 172 are supposed to carry the entire water pressure. They would be deflected, therefore, along the line $L\ C$, Fig. 6 (d), so that the combined deflection line of cantilever and arches would come into the position $A\ L\ C$. It is easily seen that this cannot be the case. The cantilever under partial load $E\ F\ K$ tends to deflect at the free end B more than the corresponding arches under full water pressure, etc., and therefore it will exert a pressure against the highest arches. In other words, certain upper arches act as if they were an additional external support for the cantilever. Consequently, such arches have to carry this load in addition to the full water pressure. There will then develop a slight adjustment between the loads of cantilever and arches, which results in the necessary coincidence of the cantilever-deflection and arch-deflection curves.

It would be difficult to determine the exact shape of the load area representing the support of the free end of the cantilever by the highest arches,* as these arches are of varying stiffness.

It is reasonable to assume that (under the pressure of the rising water) the free end of the cantilever, after having deflected from A_1 to C, Fig. 6 (d), will affect mostly the highest arch, and the ones nearer to Elevation 172 proportionately less. The load area representing the external support for the free end of the cantilever, and which is consequently also the additional pressure upon the highest arches, is therefore approximately of triangular shape, $G\ K\ N$, Fig. 6 (a).

By trial it was found that, if we assume the base $G\ N$ to be about 1 200 lb. per sq. ft. and construct the corresponding cantilever-deflection and arch-deflection

* At the crest of the Pathfinder Dam there are openings 15 ft. deep so that in this portion no arching can occur. As it is here more the purpose to give a typical example than to make a precise investigation of the statical conditions of this structure, the dam is treated as being solid from top to bottom.

lines, the two curves will fall very close together as in Fig. 6 (*f*). Therefore, within the degree of accuracy attempted here, the partial loads for cantilever and arches are represented by the areas *E F K* and *G K N* for the cantilever and *F K N G* for the arches,* Fig. 6 (*a*). The stresses in both systems may therefore be calculated in the usual way and independently of each other.

Table 4 contains the necessary figures for the construction of the cantilever-deflection and arch-deflection curves, as shown in Figs. 6 (*e*) and 6 (*f*). It should be noticed that in calculating the bending moments *M* for the cantilever, the load *G K N* acts in opposite direction to the water pressure *E F K* and tends therefore to reduce *M*.

TABLE 4.

(1) Elevation.	(2) Moment of inertia. I	(3) Bending moment, M	(4) $\frac{M}{I}$	(5) Load area, W	(6) Elevation.	(7) (8) Arch Deflections, Temperature, and Partial Load:	
						Feet.	Inches.
210	83	0	.0		210	0.0268	0.32
177	1 040	− 435 000	− 426	− 21 300	177	0.0227	0.27
153.9	3 080	− 740 000	− 244		160	0.0188	0.23
110	10 400	+ 1 960 000	+ 188	+ 20 700	110	0.0118	0.14
60	28 600	+ 16 950 000	+ 593		60	0.061	0.07
0	69 300	+ 64 260 000	+ 927	+ 45 600	0	0.0	0.0

The deflection lines for cantilever and arches in Fig. 6 (*f*) do not coincide exactly with each other. Such a complete coincidence could hardly be expected in a first approximation, and with the load areas consisting of a combination of triangles only. It is easy to see that the true deflection curve must be somewhere between the two lines *A B* and *A C* shown on Fig. 6 (*f*). A slight modification of the partial load areas (for example, along the line *F Q K* instead of the straight line *F K*, Fig. 6 (*a*)), would lead to a still closer approximation, but without changing the general results materially.

A discussion of the results of the above investigation leads to very interesting conclusions, as follows:

1.—It is seen that the very simple method for determining the relative amount of water pressure carried either by the vertical cantilever or by the horizontal arches, as outlined on page 24, leads to a fairly close approximation, at least for the lower portions of a dam.

2.—It is proved that certain arches near the crest of high dams evidently are subject to more load than full water pressure alone.

3.—The theoretical amount of load carried by the lower horizontal arches under the temperature conditions assumed is much smaller, and the stresses in the highest arches are much larger, than present practice assumes.

4.—The load on the cantilever, even under consideration of the extra support near the "free" end, is too large to be supported elastically, particularly if the

* This distribution of load between cantilever and arches compares very favorably with that obtained analytically by Mr. Wheeler.

temperature should drop more than 15° Fahr. Therefore the cantilever will fail in gravity action and produce cracks, either between the dam and foundation or in the masonry itself.

Into such open cracks water is forced under the full head and thus the statical conditions of the structure are changed entirely. The cantilever, which originally had to be considered fixed at the base, is acting now as a hinged and only partly restrained vertical beam, leaning against the horizontal arches. Certain assumptions with regard to the depth of the open cracks into the dam body, etc., would make it possible to determine approximately the division of the water pressure between vertical beams and horizontal arches, in a manner similar to that previously outlined. It probably would be very difficult to determine by calculation the actual stresses in the toe of such a dam with cracked cantilever. Theoretically, there would not be any difficulty in doing this if the depth of the open cracks and the corresponding size of the uplift at the heel were known. On the other hand, wedge action, shear, torsion, and particularly vertical arching, all interfere to such a degree with pure cantilever action in the lower portions of the dam, that any close mathematical determination of the involved stresses is problematical.

In the same manner as shown for the case of reservoir full at the time of low dam temperature, any other combination of load and temperature conditions may be investigated. Very heavy bending stresses may occur in the vertical cantilever when the dam temperature is high and the reservoir is not quite full. As a structure in general is only as strong as its weakest part and under the most unfavorable circumstances, the design of an arch dam must be made by considering different load and temperature conditions.

The methods here given will permit engineers to design curved dams on a more scientific basis than usually has been done up to the present time. The conditions already disclosed for the case of the Pathfinder Dam (particularly with regard to broken cantilever and additional pressure upon the highest arches), presumably occur similarly in every existing dam of considerably reduced gravity section, from the Shoshone Dam in Wyoming to the old Bear Valley Dam in Southern California, and from the famous Zola Dam in France to the latest arch dam in Australia. Although none of these dams has failed thus far, it is nevertheless questionable if arch dams should be designed in the future in the old way by means of the "cylinder formula."

It is a fact that in the case of such a dam we may assume that the structure probably will not fail; but we can prove indisputably that the stresses in the dam body and upon the foundation differ widely from those obtained with the old method. Such conditions are very unsatisfactory, from the point of view of both economy and safety. Further investigations necessarily must lead to a new type of arch dam, in which the cantilever and arches act together harmoniously and where all excessive tensile stresses are taken care of by a sufficient amount of steel reinforcement. In such dam design it will be necessary to assume an approximate section, determine the division of the water load between cantilever and arches, and compute accordingly the maximum and minimum stresses.

To obtain a first approximate dam section, it may be assumed that the dam acts by "arching" only. The real stresses then are determined by applying the

principles as previously outlined and the cross-section of the dam eventually is changed and reinforced with steel bars until the desired results are obtained.

In addition to the foregoing investigation of the cross-section of the dam near the arch crowns, in general it will be desirable to investigate also the sections at about the quarter points of the highest arches, for which the cantilevers may in many dams be considerably shorter. The same principles as above apply to this case, but it has to be considered that the arch-deflection curve follows a somewhat different law than at the crown. Nevertheless, the respective deflections of every horizontal arch slice in any particular vertical or inclined section may easily be determined; and therefore also the cantilever and arch stresses due to their respective water load.

Stresses in Arch Dams Resulting from a Change of Temperature.

If the temperature of a curved dam changes, the horizontal arches are shortened or lengthened accordingly, and the crown has the tendency to be moved in an up-stream or down-stream direction. The stresses involved may reach very high values,* and ought to be considered in the design of all curved dams. A difference of 40 to 50° Fahr. between winter and summer temperature,† will probably occur in most slender arch dams, if the thickness near the base is not so great that the equalizing effect of the more or less constant temperature of the earth crust is felt to a marked degree, as is doubtless the case in dams of gravity section.‡ The change of 40 or 50° may not occur all through the body of the dam, but as the change near the surface certainly is much greater than 50°, the values mentioned may well represent a fair average, at least in the higher portions of the structure. In places where a slender dam in summer or in winter is exposed for a number of consecutive days to the extremes of the atmospheric temperature, the differences in the average dam temperatures between summer and winter may be considerably higher, and hence also the stresses involved. Records of accurate measurements of temperature effects in existing slender arch dams are most desirable.

The extremes of the changes of the surface temperature of dams (due, for example, to the daily fluctuations of the atmospheric temperature), in general are much higher and may even produce surface cracks, but such stresses usually are of much less importance than those produced by a change of temperature throughout the dam, even though this change may be considerably smaller.

The presence of the large body of impounded water has a very beneficial influence upon the temperature of a dam, as the water tends to equalize any rapid and extreme changes. It will be fair to assume that the average temperature of slender dams is not far from the temperature of the impounded water.§ During the summer, the lower layers of the water in the reservoir will be considerably cooler than the upper ones, especially in deep reservoirs, and in winter there may occur

* See also Wisner and Wheeler "Investigation of Stresses in High Masonry Dams of Short Spans", *Engineering News*, August 10th, 1905, and L. J. Mensch, *Transactions*, Am. Soc. C. E., Vol. LXXVIII, p. 610.

† Mr. Wisner assumes that in the upper portions of a dam the seasonal range of temperature may exceed 100° Fahr. See "Irrigation Engineering", by H. M. Wilson, Sixth edition, p. 415.

‡ Compare conditions of Arrowrock Dam, *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 1225.

§ It is a well known fact that many dams "sweat" at the down-stream face, when the reservoir is full. In other words, water is passing constantly through the dam body. The influence of the water temperature upon the temperature of the dam body, therefore, is felt much quicker than if the transmission of heat had to be by conductivity of dry masonry only.

the reverse condition so that the difference between the extreme temperatures of the upper and lower parts of the dam throughout a whole year, always will be considerably smaller at the base than at the top* and therefore also the deformations. Furthermore, the friction between dam and foundation will prevent any movement of the base of the dam and probably for quite a distance up into the dam body, so that no bending moments and extreme stresses (and therefore also no cracks) can develop. The unyielding rock foundation acts like a kind of reinforcement for the lower portions of the dam, and this is one of the main reasons why in most dams the vertical cracks do not extend down to the bottom, but leave a core of uninjured masonry of the well known form.† Therefore it may be proper to assume that the temperature change in a masonry dam, or its influence, varies uniformly from a maximum at the top of the dam to zero at the bottom. In most cases a design assuming a maximum change of temperature of $\pm$ 25° Fahr. at the top of the dam will be safe enough, if the dam has been closed at an average temperature of about 50 to 60 degrees.

The stresses which are produced in every horizontal arch slice by a change in temperature are best calculated by some method used in arch bridge design.‡ It was assumed for the development of the formula for the deflection of a dam that the arches are free to turn at their abutments. This is not entirely correct, but the error involved for the calculation of deflections is small in most cases. For the investigation of temperature stresses, consider the arches as fixed, with unyielding abutments. The theory of the hingeless elastic arch has been highly developed for bridge design, and the same principles, somewhat modified, may be applied to the arching parts of curved dams. It would lead too far to give here all the details. The reader is referred to any book on modern arch bridge design.

A hingeless arch is three times statically indeterminate, that is, the vertical and horizontal components V and H of the arch thrust and the moment M at one abutment cannot be determined by the methods of elementary statics. Elastic deformations (which occur in the arch under the influence of external loads) lead to the so-called three equations of elastic equilibrium which, based on the principle of least work, furnish the means to determine all the unknown quantities. The following is a short outline of the theory of the elastic arch§ as first developed by E. Moersch, formerly Professor at the Swiss Federal Polytechnic in Zurich.

A hingeless arch may be considered as a cantilever beam, fixed at one end, and upon which at the other end are acting the three unknown reactions H, V, and M so as to keep the free end in place as if it were fixed, Fig. 7 (a).

It is rather difficult to determine in a direct way the quantities H, V, and M. A simple trick allows a very clever solution of the problem. By transferring the system of co-ordinates from the cantilever end O of the arch to a certain point O', Fig. 7 (b), on the axis of symmetry of the curved cantilever (for symmetrical arches only), a number of terms in the elastic equations may be made to equal zero, and this permits a simplified solution of the equations. The point O' has

* Compare the thermocline studies at the Kensico Reservoir by F. E. Hale and J. E. Dowd, *Journal of Industrial and Engineering Chemistry*, April, 1917.

† M. Merriman, "American Civil Engineers' Pocket Book", Third edition, p. 1025.

‡ F. P. McKibben, "Arches" in Taylor and Thompson's "Concrete, Plain and Reinforced", Third edition, p. 730.

§ *Revue Polytechnic Suisse*, Zurich, 1906.

to be considered as fixed inelastically to the free cantilever end of the arch, and the two forces H and V and a certain moment M', all acting in O', will keep the free end of the arch as if it were fixed as before.

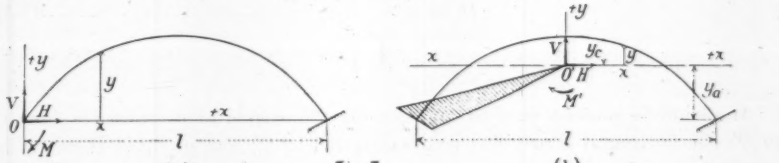

Fig. 7.

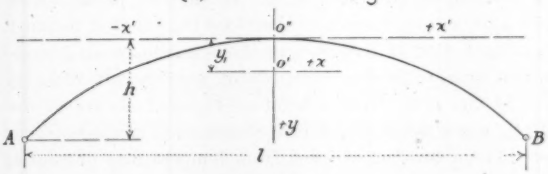

Fig. 8.

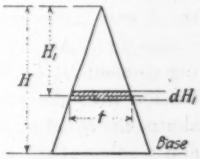

Fig. 9.

The condition for the determination of the position of O' in the case of arch dams, is that O' shall be the center of gravity of the arch. Therefore, O' will lie on the center line of the arch, at a point for which $\int_{-\frac{l}{2}}^{+\frac{l}{2}} y \, d\, L = 0$, where $d\, L$ represents the infinitesimal length of the arch. It may then be shown that a change of the length of the arch produces a horizontal thrust H of

$$H = \frac{E\,(\Delta\, L)}{\int_{-\frac{l}{2}}^{+\frac{l}{2}} \frac{y^2\, d\, L}{I}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

where

$\Delta\, L$ = change of the length of the arch, due to any cause;
l = chord of arch;
y = ordinates of elements $\Delta\, L$ of arch with regard to system of co-ordinates through point O';
I = moment of inertia of the arch slice under investigation.

In the case of a change in temperature of $\pm\, T$ degrees, $\Delta\, L$ equals $c\, T\, L$ and the corresponding temperature thrust is

$$H_t = \pm \frac{E\, c\, T\, L}{\int_{-\frac{l}{2}}^{+\frac{l}{2}} \frac{y^2\, d\, L}{I}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15a)$$

where

c = coefficient of linear expansion;
T = change of temperature, in degrees Fahrenheit;
L = length of the arch.

In the case of circular arch dams, the moment of inertia, I, of a horizontal arch slice is constant throughout. Equation (15a) therefore may be written

$$H_t = \pm \frac{E\,I\,c\,T\,L}{\int_{-\frac{l}{2}}^{+\frac{l}{2}} y^2\,d\,L} \qquad \text{.....(15}b\text{)}$$

It should be noticed that, by transferring the system of co-ordinates from O to O', the horizontal thrust also was transferred to O', and produces therefore in any arch section a bending moment of the value $H_t\,y$. In the two places where the X axis intersects the arch axis, y equals zero and therefore the bending moment is also zero. An increase in temperature of T degrees produces at the crown a bending moment $M_c = H_t\,y_c$, which tends to produce compression near the intrados and tension near the extrados. At the same time a bending moment occurs at the abutments equal to $M_a = H_t y_a$, which tends to produce compression at the extrados and tension at the intrados. These bending stresses from temperature, of course, have to be combined with the direct arch thrust due to water pressure.

In case the reservoir is empty or nearly so, at a time when the temperature of the dam is considerably above the closing temperature, the arches are lengthened and the crown will move in an up-stream direction. This produces bending moments at the crown which in many existing unreinforced dams are of such magnitude as to crack the concrete in the tension zone, sometimes for several feet in depth. When the water in the reservoir rises and the temperature also falls, the arch crowns are forced back, and bending moments are produced which are opposite in sign. Then the concrete probably will fail in tension for a considerable depth, this time at the down-stream side, and there is left a comparatively thin core of uninjured concrete, upon the strength of which alone the safety of the whole structure ultimately may depend. Most fortunately, the vertical cantilever action of every dam tends to reduce the size of the bending moments due to temperature, etc., and therefore also the extreme stresses; but the more a dam depends upon arch action, the more important will be the question of the temperature stresses.

To make use of the Equation (15b) for the determination of the arch thrust due to changes in temperature, it is necessary to know the equation of the center line of the arch. In the case of curved dams the center line in general is a circle. The determination of the integrals involved requires too much work for practical calculations. If we assume a parabola instead of a circle for the center line of the arches, the calculations lead to very simple formulas and the error involved is very small for arches with a central angle of not more than 100 to 110 degrees.

The equation of a parabola $A\,O''\,B$, Fig. (8), in rectangular co-ordinates, the origin being at the vertex and the axis of the parabola being the axis of Y, is

$$x^2 = 2\,p\,y \qquad \text{.....(16}a\text{)}$$

Let h be the height of the parabolic arc and l the span length $A\,B$.

For $x = \frac{l}{2}$, $y = h$, and

$$\therefore 2p = \frac{l^2}{4h}$$

$$x^2 = \frac{l^2}{4h} y$$

$$y = \frac{4h}{l^2} x^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16b)$$

To find the distance of the center of gravity of the arc $A\ O''\ B$ from the vertex O'', we have the condition

$$\int_{-\frac{l}{2}}^{+\frac{l}{2}} (y - y_1)\, d\,L = 0$$

or

$$\int_{-\frac{l}{2}}^{+\frac{l}{2}} \left(\frac{4h}{l^2} x^2 - y_1\right) d\,L = 0$$

In case of a flat arch, approximately, $d\,L = d\,x$ and the solution of Equation (16b) gives

$$y_1 = \frac{1}{3} h.$$

In transferring the origin of the co-ordinates from O'' to O' the Equation (16b) of the parabola changes into

$$y + y_1 = \frac{4h}{l^2} x^2$$

or

$$y = \frac{4h}{l^2} x^2 - \frac{1}{3} h.$$

The denominator of Equation (15a), substituting $d\,x$ for $d\,L$, becomes

$$\int_{-\frac{l}{2}}^{+\frac{l}{2}} y^2\, d\,x = \int_{-\frac{l}{2}}^{+\frac{l}{2}} \left(\frac{4h}{l^2} x^2 - \frac{1}{3} h\right)^2 d\,x.$$

The integration is very simple and leads to the expression

$$\int_{-\frac{l}{2}}^{+\frac{l}{2}} y^2\, d\,L = \frac{4\,h^2\,l}{45}.$$

This value introduced in Equation (15b) gives

$$H_t = \frac{45\,E\,I\,c\,T\,L}{4\,h^2\,l^2}$$

or, approximately,

$$H_t = 11.25\,\frac{E\,I\,c\,T}{h^2} = 0.94\,\frac{t^3}{h^2}\,E\,c\,T \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

This equation makes it possible in a short time to determine with reasonable accuracy the thrust in an arch due to a change in temperature. The stresses then may be calculated in the usual way, for combined axial pressure and bending moment.

For average conditions, assume a change in temperature in the upper parts of a dam between summer and winter of about ± 25° Fahr. With $c = 0.000\,006$ and $E = 2\,500\,000$ lb. per sq. in., Equation (17) gives

$$H_t = \pm\ 0.94\ \frac{t^3}{h^2} \times 2\,500\,000 \times 144 \times 0.000\,006 \times 25$$

$$H_t = \pm\ 50\,000\ \frac{t^3}{h^2} \qquad \text{(17}a\text{)}$$

for a horizontal dam slice of 1 ft. vertical thickness (t and h measured in feet).

Equation (17a) shows that the temperature thrust H_t increases with the third power of the dam thickness. In curved dams of heavy cross-section, the temperature thrust therefore in general attains very high values. In a curved gravity dam, the cantilever action may prevent (at least partly) the temperature deformation of the horizontal arch slices which would produce dangerous bending moments, and thus reduce to a great extent the influence of the temperature to simple axial compression or tension, as in a straight dam. In this latter case a change in the dam temperature of 25° Fahr. produces stresses of about $f_c = E\ c\ T = 2\,500\,000 \times 0.000\,006 \times 25 = 375$ lb. per sq. in.

The horizontal thrust H_t which acts, it must be remembered, at a distance of one-third of the rise h from the crown, produces a bending moment at the crown of the value

$$M = H_t \times \frac{h}{3} = 0.31\ \frac{t^3}{h}\ E\ c\ T$$

The temperature thrust, H_t, and the bending moment, M, combined produce the stresses

$$f_c = \frac{H_t}{t} \pm \frac{M}{\text{Sect. Mod.}} = \frac{0.94\ \frac{t^3}{h^2}\ E\ c\ T}{t} \pm \frac{0.31\ \frac{t^3}{h}\ E\ c\ T}{\frac{1}{6}t^2}$$

$$f_c = 0.94\ E\ c\ T\ \frac{t}{h}\left(\frac{t}{h} \pm 2\right) \text{ lb. per sq. ft.} \qquad \text{(18}a\text{)}$$

For $E = 2\,500\,000$ lb. per sq. in. and $c = 0.000\,006$ we have for every degree Fahrenheit of change in temperature

$$f_c = 2\,000\ \frac{t}{h}\left(\frac{t}{h} \pm 2\right) \text{ lb. per sq. ft.} \qquad \text{(18}b\text{)}$$

and for 25° Fahr.

$$f_c = 50\,000\ \frac{t}{h}\left(\frac{t}{h} \pm 2\right) \qquad \text{(18}c\text{)}$$

if all dimensions are measured in feet.

The upper sign in Equations 18 (a), 18 (b), and 18 (c) gives the stresses at the intrados and the lower sign at the extrados, if (+) stands for compression and (—) for tension. The change of the temperature of course has to be given the proper sign, *i. e.* (+) for a rise and (—) for a lowering for T degrees Fahr.

The bending moment at the arch abutments would be double the size of those at the crown if the arches really were entirely fixed, as the lever arm of H_t is double. But no tension can occur in the masonry because the bond between rock and masonry will be destroyed as soon as tension tends to develop. On the other hand, the compression stresses will be considerable, but the total compression force of course never can be greater than the arch reaction due to the external horizontal forces.

For the investigation of the temperature stresses in the case where the reservoir is empty, we have to assume (for obvious reasons) that the horizontal arches will turn freely at the abutments, or nearly so, unless the dam is anchored with steel bars to the rock foundation. The calculation of the temperature stresses in this case, then, preferably is based upon the theory of the two-hinged arch.*

Stresses Due to the Shortening of the Horizontal Arches Under Water Pressure.

In general the stresses in circular arch dams are computed by the simple method of calculating the axial arch thrust, as the product of radius of extrados and water pressure. This is only an approximation. Under the influence of the axial thrust, an arch ring tends to shorten itself and to decrease radius and span. As the abutments are fixed (span not changed), certain internal stresses similar to those due to a decrease in temperature are produced, besides the ones due to centric arch pressure. It can be shown by means of the theory of the elastic arch as outlined previously, that this shortening of the arch produces a negative horizontal thrust H_f like a fall in temperature, and acting in the same elevation (Point O' in Fig. 7 (b)) as H_t. Therefore, we may write (Equation (14):

$$H_f = \frac{E \, \Delta L}{\int_{-\frac{l}{2}}^{+\frac{l}{2}} \frac{y^2 \, d \, L}{I}}$$

The denominator is the same as before; ΔL is the shortening of the arch due to that part of the water pressure which the arch slice under consideration is supporting.

There is

$$\Delta L = - \frac{P \, R_u}{A \, E} L$$

(arch is shortened under the influence of the water pressure; therefore, ΔL is negative),
and

$$\int_{-\frac{l}{2}}^{+\frac{l}{2}} \frac{y^2 \, d \, L}{I} = \frac{4 \, h^2 \, l}{45 \, I}$$

* F. P. McKibben, "Arches" in Taylor and Thompson's "Concrete, Plain and Reinforced." Third edition, p. 730.

where

P = water pressure per square unit, supported by arch;
R_u = up-stream radius;
L = length of arch slice;
t = thickness of arch slice;
A = Area of horizontal arch slice, 1 ft. high;
I = moment of inertia of the horizontal arch slice.

All dimensions are to be given in feet.

$$H_r = - \frac{E \frac{P R_u}{E A} L}{\frac{4 h^2 l}{45 I}}$$

$$H_f = - 0.94 P R_u \frac{t^2}{h^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

and, because

$$\frac{P R_u}{t} = f_c \text{ (lb. per sq. ft.)}$$

$$H_f = - 0.94 f_c \frac{t^3}{h^2} \text{ lb.} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19a)$$

The bending moments and stresses due to H_f are calculated in the same manner as described previously for temperature deformations.

Stresses Due to Shrinkage of the Concrete.

Exactly the same theory as above holds good for the effect of the shortening of a concrete arch due to shrinkage during the setting and hardening of the cement. The reason for this shrinkage is not quite clear. One possible explanation lies in the fact that the chemical heat produced during the process of hardening increases the temperature of the entire mass for many degrees, especially if the cubical dimensions of the structure are considerable; and this produces a corresponding expansion of the material. When the chemical heat is lost again (in big masses this may be only after years), the concrete has hardened and the reduction of its temperature produces tensile stresses resulting in the well known "shrinkage" cracks, which, combined eventually with deformations due to the influence of the outside temperature, may be very considerable and in many cases rather dangerous.

The shrinkage shortening is different for varying mixtures of concrete, and in general is not less than $\frac{1}{4}$ in. per 100-ft. length. This corresponds to a decrease in temperature of about

$$T = \frac{\Delta L}{c L} = \frac{0.25}{0.000\,006 \times 100 \times 12} = 35° \text{ Fahr.}$$

In a straight gravity dam, the shrinkage of the concrete produces tension in the direction of the axis of the dam. Sometimes the stresses are so high as to exceed the tensile strength of the concrete, so that visible cracks occur. These may be without material influence upon the safety of a gravity dam; but the conditions are very different in the case of a curved dam, which depends for its stability upon

horizontal arching. In high and slender arch dams, the tension produced by shrinkage may be able to force the arches (and therefore also the free end of the cantilever) sufficiently in a down-stream direction so that a corresponding shortening ΔL_s of the arches may occur without producing such excessive tensile stresses as would result in dangerous cracks. But even in such a very favorable case, the shortening of the arches produces secondary stresses of the same nature as a decrease in temperature. A negative horizontal thrust is produced, using Equation (14), as follows:

$$H_s = \frac{E(\Delta L_s)}{\int_{-\frac{l}{2}}^{+\frac{l}{2}} \frac{y^2\, d\, L}{I}} = -\frac{E(\Delta L_s)}{\frac{4\,h^2\,l}{45\,I}}$$

$$H_s = -0.94\,\frac{E\,\Delta L_s}{l}\,\frac{t^3}{h^2} \text{ lb. (dimensions in feet)} \ldots\ldots (20)$$

The stresses resulting from this thrust, which acts outward, are calculated in the same manner as temperature stresses.

It is evident that these secondary stresses from temperature, rib shortening and shrinkage also may be obtained by calculating the size and position of the arch thrust. Whenever tension occurs at a point, this indicates that the resultant arch thrust near that point falls outside the middle-third of the arch section.

A complete investigation of the stresses in arch dams should cover the following items:

After the principal dimensions (as height, radii of curvature, etc.) have been determined to suit local conditions, first compute for an assumed cross-section of the dam the probable division of the water load, ice pressure, etc., between cantilever and horizontal arches. This calculation should include consideration of shrinkage and low temperature, and also the conditions as they exist when the dam temperature is at the seasonal maximum. For each case the extreme direct stresses both in cantilever and in horizontal arches are determined; and the dimensions of the assumed cross-section of the dam are changed and the calculations repeated, until reasonable values are obtained for the resulting direct unit stresses.

The second part of the design consists in the determination of the secondary stresses due to changes in the length of the arches as a result of direct axial pressure,* temperature, and shrinkage.

The extreme stresses in any part of the dam are calculated by combining the direct and secondary stresses, in the most unfavorable combinations that they possibly can occur. For both cantilever and arch actions of the dam, the conditions of reservoir full, ice pressure, shrinkage and low temperature probably will produce the maximum stresses; while, on the other hand, the conditions of reservoir empty and dam temperature high may develop reversed stresses provision for which must also be made.

The design of an arch dam therefore shows in many respects a great similarity to the design of a concrete arch bridge, and the work involved is about the same. A dam may be considered as a permanently safe structure if the design as outlined

* See also p. 43.

shows at no point of the dam excessive compression, and if the eventually occurring tensile stresses are taken care of by sufficient steel reinforcement.

Effect of Vertical Cracks.

It is a well known fact that many long dams (either curved or straight) have shown vertical cracks when the reservoir is not filled. These cracks are due mostly to the shrinkage of the concrete during the time of setting and hardening, and partly to an occasional lowering of the dam temperature. When the reservoir is filled, the cracks in general are closed under the influence of the water pressure. As soon as the reservoir is partly or entirely emptied, the cracks open again.

It has been noticed in many dams that vertical cracks occur particularly at places where there are abrupt changes in the slopes of the hillsides. This is easily explained by the fact that (for example, with a rise in the dam temperature), the central and higher parts of a dam expand in a vertical direction considerably more than the portions which are not so tall part way up the hillsides. It is understood, of course, that the relative position of the bed-rock at the bottom of the canyon and along the abutments is not changed by the seasonable temperature changes of the earth's crust.

Take as an example a dam of height $H = 250$ ft. A fall in the average temperature of the dam body of, say, 10° Fahr., tends to lower the center parts of the dam in a vertical direction about

$$\Delta H = c\,T\,H = 0.000\,006 \times 10 \times 250 = 0.015 \text{ ft.} = 0.18 \text{ in.}$$

The same change in temperature lowers a vertical dam slice near the side-hills, where the dam may be only 50 ft. high, about

$$\Delta H' = 0.000\,006 \times 10 \times 50 = 0.0003 \text{ ft.} = 0.036 \text{ in.}$$

so that the crest of the dam has been lowered in the center $0.18 - 0.036 = 0.144$ in. more than nearer to the side abutments.

Such a tendency of vertical temperature deformations in a dam body produces heavy shearing stresses, and this is an explanation for the fact that often cracks are formed in vertical planes located without regard to joints, passing, in some cases, through headers within 2 or 3 in. of their edges.* Therefore it may happen that (in case of an abrupt change in the slope of the hillside) a decrease of the dam temperature will produce a vertical crack, due to direct horizontal tension and vertical shearing stresses combined. At the same time there is the possibility that the middle portion of the dam will become slightly displaced in a vertical direction with respect to the adjacent portion, which may be of considerably smaller height and therefore shrink less. If an arching curved dam under such conditions comes under pressure due to a sudden rise of the water, and before the dam temperature has been changed again, the two surfaces of the crack are displaced somewhat with respect to each other, so that the arch pressure may result in serious injury to the irregular surfaces of the cracks.

It may also happen in certain dams that the pressure of the rising water will deflect the middle portion of the dam somewhat in a down-stream direction, even before the water has reached the foot of the less tall section farther up-hill. This

* "American Civil Engineers' Pocketbook", Mansfield Merriman, Third edition, p. 1025.

too would result in a relative displacement of the two surfaces of the crack from their original position, with the same consequences.

The opening and closing of construction joints and cracks, which in certain arched dams are ¼ to ⅜ in. wide, is always a very serious matter. A vertical crack in a dam, which for its stability depends more or less on arch action, is a source of decided weakness, as considerable compression stresses cannot be safely transmitted across irregular cracks. The danger of gradual disintegration is particularly great if such cracks open and close periodically. When a crack in a dam has opened, small particles may become loosened and fall down to some place where the crack gets narrower and holds the small fragments. More may follow, especially under the influence of rain or frost. When the arch again comes under the pressure of the rising water in the reservoir and tends to close the crack, local accumulations of loose material will prevent this partly. Practically all the arch thrust therefore will be concentrated in these spots, and a tremendous local pressure is thus produced. The loose concrete particles probably will be crushed to dust, but at the same time they will also stress the adjacent concrete beyond its ultimate strength. Thus, weak spots are produced, and after a number of repetitions of this process, which is of rapidly increasing magnitude, the concrete will crumble and the structure may collapse. But without going that far, it is easy to see that by such local concentrations of the arch thrust great eccentricities with regard to the assumed line of pressure (approximately the center line of the arches) may occur, thus producing bending moments and secondary stresses of unknown magnitude. The writer wishes to leave the question open, if such a cracked structure (which at certain times has to sustain reversed stresses) can be considered safe and for how long. Probably few engineers would trust a cracked member in any kind of a structure, but this seems to be done by many in the case of curved masonry dams.

It has been suggested that grout be forced into these contraction cracks at a time when the temperature is low and the reservoir empty. As it will be impossible to fill evenly and over their entire area cracks of ⅛ in. width and less (and which may extend through the entire thickness of the dam, that is, often from 30 to 50 ft.) one thus would produce artificially just what ought to be avoided, that is, an uneven distribution of the stresses over the arch section which, in a dam that depends for its stability on arch action, sooner or later will result in disaster. The writer wishes to sound a special note of warning against such a practice, which offsets all the advantages of a good and safe design. Even if a dam treated in such a way does not fail, the factor of safety is lowered to an unknown degree.

On the other hand, if a dam is closed at a low temperature (either intentionally or unintentionally during construction, or as mentioned above by artificial means), a rise in temperature will produce bending moments, which eventually will cause other and perhaps still more dangerous cracks.

In a dam which depends for its stability mainly on gravity action, although curved in plan, temperature and shrinkage stresses, of course, do not have the same importance as in other arch dams. In such cases cracks may be closed by grouting, for the sake of good appearance or to improve the water-tightness, without any important statical disadvantage.

Vertical Arching.

As indicated before, it cannot be assumed that a gravity dam across a comparatively narrow canyon acts in the same manner as a dam which is long compared with its height. There can be no doubt that vertical arching occurs from one side of the canyon to the other,* and this action falsifies more or less the general assumption of pure gravity action. A vertical dam slice, due to its own weight, will deform elastically in a vertical direction. Certain internal vertical arches in the dam (which may be imagined to span from one side of the canyon to the other) also will deflect in a vertical direction due to their share in supporting the weight of the dam. The problem of finding the exact amount of the weight of the dam which is supported by a vertical arch action is a very complex one, and can be solved only by approximation. In any case it is not practicable to try to develop general formulas, which become very complicated.

It will be demonstrated here that for any special case it is possible to compute (with a fair degree of accuracy) how much the vertical arch would deflect under the assumption that it has to carry all the load. On the other hand, it is easy to determine how much the vertical dam slices in the middle of the dam would be shortened in a vertical direction, due to their own weight, if no vertical arching did occur. By comparing the two deflections one can judge approximately what percentage of the weight of certain portions of the dam will be carried either by vertical column or by vertical arch action.

Assume the dam to be of theoretical triangular cross-section. An isolated vertical slice of the dam, Fig. 9 (see p. 27), if no other forces but gravity are acting upon it, will shorten in a vertical direction due to its own weight, as follows:

$$\Delta H = \int_0^H \frac{\text{Stress}}{\text{Modulus of Elasticity}}\, d H_1$$

where H is the height of a particular vertical dam slice 1 ft. thick. If t is the thickness of the dam at an elevation H_1 below the crest,

$$\text{stress} = \frac{w \dfrac{t H_1}{2}}{t} = w \frac{H_1}{2}$$

where w is the weight of a cubic unit of concrete.
This gives

$$\Delta H = \int_0^H w \frac{H_1\, d H_1}{2 E} = w \frac{H^2}{4 E} \qquad (21)$$

Assume now the special case that the dam is built across a canyon which is rather narrow at the bottom and in which the hillsides have a slope of about 45 degrees. No water pressure will be considered at present. This example will cover, more or less, many practical cases and at least indicates in what respects present methods of dam design may have to be modified.

The vertical arching will occur along the lines of greatest resistance, that is, for which the vertical deflection is a minimum. The laws of statics indicate that for uniform vertical load these lines of pressure are parabolas. The center line of the bundle of elementary arches may be assumed as the axis of the vertical arch.

* See also *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 622.

To simplify the calculations, replace the parabolic arch by a circular one, the two being nearly identical in this case. The vertical arch therefore will be inclosed between two co-axial cylinders, the axis being in a horizontal plane near the foundation of the dam, and about in the center of the canyon. The radii of these cylinders will be chosen such that the larger cylinder touches the crest of the dam and the smaller one spans across the bottom of the canyon. The center line of the vertical arch goes through the center of gravity of its cross-section, as indicated in Figs. 10 (*a*) and 10 (*b*).

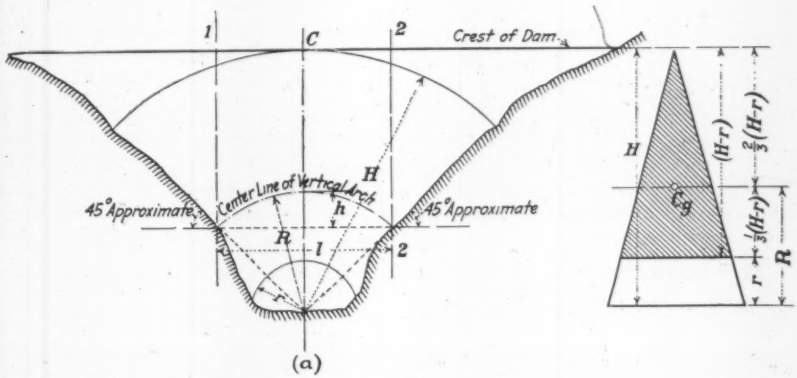

FIG. 10.

The notation used is as follows:

H = radius of extrados of vertical arch (approximately height of dam);
r = radius of intrados of vertical arch;
R = radius of center line, or axis of vertical arch;
l = span (of axis) of vertical arch across the canyon;
h = rise (of axis) of vertical arch;
D = crown deflection of vertical arch;
L = length (of axis) of vertical arch;
ΔL = shortening (of axis) of vertical arch due to the arch thrust;
W = weight of that part of the dam which may act upon the vertical arch (inclusive of the weight of the arch);
A = area of cross-section of vertical arch (assumed to be constant and equal to the area of the cross-section at the crown);
N = arch thrust;
w = weight per cubic unit of masonry;
E = modulus of elasticity.

As weight W consider only that part of the dam which lies between the vertical planes 1-1 and 2-2 through the ends of the center line of the vertical arch, as the weight of the portions to the left of 1-1 and to the right of 2-2 will have no influence upon the shortening of the axis of the arch.

The crown deflection of the vertical arch is, by Equation (11),

$$D = \frac{3}{16}\,\frac{L}{h}\,\Delta L$$

Further

$$\Delta L = \frac{N\,L}{A\,E}\,;\; N = \frac{W\,l}{8\,h}\ \text{(approx.)};\; W = w\,A\,l.$$

Therefore

$$\Delta L = \frac{w\,l^2\,L}{8\,h\,E}$$

and

$$D = \frac{3}{128}\,\frac{w\,l^2\,L^2}{E\,h^2}$$

For this case, approximately, the following values for l, L, and h can be used:

$$l = R\,\sqrt{2}\,;\; L = \frac{r}{2}\,R\,;\; h = 0.3\,R$$

Therefore

$$D = 1.29\,\frac{w}{E}\,R^2.$$

The mean radius R of the vertical arch equals $\frac{H+2\,r}{3}$. The crown deflection, then, is given by the equation:

$$D = 0.143\,\frac{w}{E}\,(H+2\,r)^2.$$

This deflection, D, taken theoretically, is to be measured at the crown of the center line of the vertical arch; and the portion of the dam above, in addition, will settle approximately the same amount as a free column. The height of the dam portion above the crown of the center line of the vertical arch is $\frac{2}{3}\,(H-r)$; the additional settlement of point C, Fig. 10 (a), perpendicularly above the crown, is according to Equation (21):

$$D' = \frac{w}{4E}\left[\frac{2}{3}\,(H-r)\right]^2 = 0.11\,\frac{w}{E}\,(H-r)^2$$

and the total deflection, D_a, of point C is

$$D_a = D + D' = 0.143\,\frac{w}{E}\,(H+2\,r)^2 + 0.11\,\frac{w}{E}\,(H-r)^2 \ldots\ldots\ldots (22)$$

if pure arch action only would occur.

If pure column action only would occur, the deflection, D_c, of point C is

$$D_c = 0.25\,\frac{w}{E}\,H^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

The weight supported either by vertical column or vertical arch action is about inversely proportional to the corresponding deflection of identical points in both systems; Equations (22) and (23) therefore furnish the means to estimate approximately how much of the weight of the center portion of such a dam is supported by true gravity action and how much is thrown upon the hillsides by horizontal arching.

Consider now the special case for which the canyon is so narrow at the bottom that in Equations (22) and (23) we may neglect r, with rough approximation, as being very small as compared with H.

Then:

$$D_a = 0.143 \frac{w}{E} H^2 + 0.11 \frac{w}{E} H^2 = 0.253 \frac{w}{E} H^2 \ldots\ldots\ldots\ldots (24)$$

This value compared with Equation (23) shows that the vertical arch deflection for point C is approximately just as much as the settling of the column below C would be if no vertical arching occurred.

This seems to indicate that for certain parts of such a dam near the center, about one-half of the weight is supported by vertical arch action and thus is thrown upon the hillsides. Therefore only half of the weight would act perpendicularly upon the base. In the case of a lowering of the dam temperature, evidently a still greater percentage of the load is thrown upon the vertical arch.

The foregoing calculated distribution of load between pure gravity action and vertical arching is only a very rough approximation. The value of the whole investigation lies principally in the mathematical proof that vertical arch action really may interfere with pure gravity action; and this apparently the more the higher the dam in comparison with the width of the canyon, and the steeper the side slopes.*

Vertical arching probably exists in the same proportion for the transmission of compressive stresses when the reservoir is full, that is, the compressive stresses in the vertical cantilever do not act entirely upon the base directly but in part are thrown upon the hillsides by vertical or inclined arching. Thus, beside making a gravity design of such a dam a very uncertain matter, vertical arching, combined with true cantilever action, may produce near the foot of the side-hills or abutments compressive stresses which are considerably greater than those assumed as limits for the standard gravity design. The factor of safety of the structure, calculated by the theory of gravity action, thereby would be decreased correspondingly, although ultimate collapse after initial failure may be prevented by shear strength and by friction along the hillsides.

In the case of curved dams of long radius and heavy cross-section, the conditions in general are still worse. The vertical arch is loaded eccentrically, due to the up-stream curvature of the dam. This produces an uneven distribution of the vertical arch stresses, and therefore causes still greater concentration of stresses at certain points. It also should be noticed that in addition to the pressures due to vertical arching and to cantilever action, certain parts of the foundation along the side-hills eventually have to sustain the thrust from horizontal arching, especially due to an increase in temperature. The three compression forces (from cantilever action, and from vertical and horizontal arching) act in different directions, but may be combined in the one resultant which produces unit stresses which at certain points of the dam will be considerably above the safe unit stresses assumed in design. Gravity dams across narrow canyons therefore cannot be safely

* Considerable vertical arching certainly exists in both the Pathfinder and Shoshone Dams, both of which are of great height and span very narrow canyons. Similar conditions are found in the Lake Cheesman Dam and in others, and the calculations with regard to gravity action of these structures certainly ought to have been modified accordingly.

considered to depend for their stability on gravity action alone, and the methods of design should be changed accordingly.

It lies in the power of the designing engineer to give the dam such a shape that (with due consideration to all the facts as already outlined) the unit stresses in dam or in foundation shall be at no point above the allowable stresses. These considerations necessarily will require a different distribution of material for strengthening weak points and for saving unnecessary material in places where (in the ordinary design of gravity dams) it would be pure waste, or still worse.

An investigation along these lines therefore will lead very often to the adoption of a curved or a hollow type of dam. Nevertheless, many engineers of the old school still favor the straight gravity dam. This is the more astonishing as such a structure (even in its most favorable field of application, that is, across a wide valley where no vertical arching is possible) in general has a factor of safety of less than 2.

According to standard practice, a gravity dam is considered as safe, if the resultant of all the forces lies within the middle-third of the section, if the maximum compression produced does not exceed the allowable pressure upon the foundation; and if there is sufficient safety against sliding. A dam designed according to these principles generally is assumed as having a factor of safety of not less than 2.*

This assumption is not quite correct. Any straight dam in which the resultant with reservoir full passes through the lower middle-third, will have tension produced at the heel as soon as the water load is increased somewhat.† An increase of 30 to 50% in most cases will open a crack between masonry and foundation at the up-stream side of the dam, and the resulting uplift due to the water entering into the crack under the full head H, together with the increased load, may produce excessive compression at the toe, thus permitting the crushing either of foundation or of masonry. This fact seems to have been very often overlooked, but it easily can be proved for every standard profile of gravity dam.

Still another point is to be considered. As soon as a crack opens at the heel of the dam and admits water, the decreased area of contact between dam and foundation and the reduced weight of the dam due to uplift, may make it possible that the friction at the foundation may be overcome and that the dam may begin to slide. In short dams the friction along the side abutments in general will prevent direct failure, the dam acting in part as a horizontal beam as soon as sliding has begun; but if (due to temperature or shrinkage cracks) certain vertical center slices are practically isolated, such parts may slide in a down-stream direction independently of the remainder of the dam.

Many authorities find it necessary to investigate the factor of safety against overturning. It is hard to understand why (in a scientific investigation of gravity dams) the expression "overturning" still occurs, except as a mere illustration of the effect of excessive compression at the toe. A high gravity dam can never fail by overturning unless the pressure on the foundation or certain stresses in the masonry, have long before exceeded the allowable value. As soon as the resultant of the external forces falls outside the middle-third, tension is produced at the

* E. Wegmann, "The Design and Construction of Masonry Dams", New York, 1907.

† For many dams there exists also the danger that after a number of years the reservoir will be partly filled with silt and débris.

up-stream side and the unit compression at the toe increases very rapidly. With the horizontal water pressure increased less than 50%, foundation or masonry in most cases will be crushed; and this at a time when the so-called factor of safety against overturning still is considerably above the usual limit. After crushing the foundation or masonry, the dam, of course, would readily overturn but not before. Little attention seems to have been paid to the fact that very seldom, if ever, a gravity dam has failed by overturning. Practically all failures have occurred by sliding, and this indicates that the factor of safety against sliding is still less than the one against overturning, unless the rock foundation has been prepared so as to make sliding impossible.

All these considerations prove beyond any doubt that in general the factor of safety of a pure gravity dam probably is nearer 1 than 2. This fact certainly accounts for many failures, as sometimes a small oversight in design or construction has led to collapse and disaster, which with a structure of a higher factor of safety probably would not have occurred.

It is true that the gravity type has a long record of successful application, with comparatively few failures. This probably is due, however, only to the very fortunate circumstance, that for storage dams no high percentage of overload above the assumed load ever is possible. It is a matter of fact, however, that no other engineering structure of acknowledged good design has such a small factor of safety as a pure gravity dam.

Efforts have been made to overcome some of the objections against the pure gravity type by building the dam curved in plan. It was generally assumed (though it never has been mathematically proved) that a curved gravity dam afforded a greater degree of safety than a straight one. No method was known by which to figure how much this increase in safety really is. Most investigations along these lines were based upon the assumption that horizontal arching would combine with the vertical cantilever action in supporting the water pressure, but theory and practice never seemed to agree very closely in this matter.

One great advantage of a curved dam over a straight one is the fact that slight mistakes in the assumptions regarding the quality of the foundation, the soil pressure, the uplift, etc., will affect a curved dam to a much smaller degree than they will a straight one.

On the other hand, it can be proved that no curved dam of gravity section will develop any considerable amount of horizontal arching at a time of low temperature unless it should have failed first in gravity action, or by sliding along some horizontal plane. A typical example is shown on page 50 with regard to the Lake Cheesman Dam; and the method given there and applied to any other curved gravity dam will show similar results.

Notwithstanding the fact that direct horizontal arch action in a curved gravity dam cannot be depended on to be of any help to the cantilever (at least not before the dam has failed already in gravity action), the curvature nevertheless really does increase very materially the stability of such a structure under working conditions. This is due to another property of a curved dam* which will be discussed later. (See page 44.)

It is evident that a curved dam of full gravity section and resting upon an adequate foundation, is an absolutely safe structure. True engineering, however,

* See also H. Visher and L. Wagoner, *Transactions*, Technical Soc. of the Pacific Coast, 1889.

includes not only making a structure strong enough, but also giving due regard to economy in the use of material. The numerous examples of slender arch dams which have been built with a considerable saving in cost, as compared with dams of full gravity section for the same sites, prove conclusively that the curved gravity type is not in all cases the ideal solution of the dam problem, particularly not from the point of view of economy.

Progressive designers, therefore, have tried to take advantage of the statically very economical arch action, wherever the local conditions were favorable to an arch dam. Thus there have been built many dams of greatly reduced gravity section in which doubtless considerable horizontal arching must exist to enable the structure successfully to withstand the water pressure.

Although the methods of calculating the stresses in such curved dams in general have been very crude (as but seldom the influences of temperature, shrinkage, etc., were considered), it is rather a matter of fact that no curved dam has ever failed.

The stresses in the horizontal arches of curved dams, under the assumption that no cantilever action exists, in general are determined by the simple so-called cylinder formula:

$$\text{Arch thrust} = \text{water pressure} \times \text{up-stream radius}$$

or

$$N = P R_u$$

This formula, if applied to the calculation of the stresses in arch dams, is defective in several ways. First, it indicates a maximum arch thrust near the base of the dam where, as a matter of fact, it is zero. Second, the arch thrust at the crest would appear to be zero, as there is no water pressure at that elevation, while in general the deflections of arch dams are a maximum at the crest, indicating also the occurrence of correspondingly large stresses to produce such deflections.

Further, it also is generally assumed that the axial thrust passes through the center of the arch section so that the stresses are uniform in every horizontal plane. This is not quite correct, especially for arch rings of great thickness as compared with the radius of curvature. This is easily explained by the fact that in any horizontal arch slice (which may be imagined as composed of elementary rings varying in radii from R_d at the down-stream side to R_u at the up-stream side), the rings farther down stream and with smaller radii are stiffer than those nearer the up-stream side of the dam. Therefore, they will take more load, and their unit stresses will be higher.

An easy way to prove this is to apply Equation (13) to calculate the deflections D_u and D_d of the elementary arch rings near the up-stream and down-stream sides, respectively. Thus

$$D_u = 1.56 \frac{R_u}{E} f_u$$

where f_u is the unit stress at the up-stream side and

$$D_d = 1.56 \frac{R_d}{E} f_d$$

if f_d is the unit stress at the down-stream side of that particular horizontal dam slice.

As D_u and D_d of course have to equal each other, both being necessarily equal to the deflection of the center line of the arch,

$$D_u = D_d = 1.56 \frac{R_u}{E} f_u = 1.56 \frac{R_d}{E} f_d$$

$$\therefore \frac{f_d}{f_u} = \frac{R_u}{R_d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

The stresses at the down-stream and up-stream sides of each horizontal arch slice therefore vary inversely as the respective radii.*

It can be shown that the eccentricity, x, of the axial thrust in any arch is

$$x = \frac{t^2}{6(R_u + R_d)} = \frac{t^2}{12\,R} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

This value is considerable in thick dams with short radii; for example, if $R_u = 200$ ft.; $R_d = 100$ ft.; $t = 100$ ft.,

$$x = \frac{100^2}{6\,(200 + 100)} = 5.6 \text{ ft.}$$

In dams of slender cross-section and long radius, the eccentricity of the arch thrust becomes small and eventually may be neglected.

Many curved dams of reduced gravity section have been designed by computing roughly the probable amounts of cantilever and of arch action, and then stating that "the dam would be able to withstand the water pressure, even if the horizontal arches had to support all the load." This statement seems to show caution, but does not have at all the effect desired. On the contrary, this assertion unfortunately only creates a wrong impression about the factor of safety of curved dams. It is of course admitted by every one that the dam in the lower parts (where often the thickness of the arch is greater than the span across the canyon) cannot act by arching, but does its work more like a wedge. Also, it is always out of the question under ordinary conditions that the lowest parts of a dam can move appreciably, which is the first condition for developing arch stresses.

The seemingly very innocent remarks found in many technical reports on arch dams about eventual arching in their lower parts and the apparently very safe unit stresses and correspondingly high factors of safety in practical constructions, have done much harm.

As a matter of fact, it may be proved by the methods here given that practically all dams which have been designed without any special investigations as regards combined gravity and arch action, in reality have the cantilever action at least partly destroyed. Certain of the upper arches then have to support the broken cantilever in addition to the full water pressure, and consequently these upper arches are stressed much more than the simple cylinder formula indicates. The occurrence of cracks in the cantilever is due to the fact that in most existing arch dams the arches are not stiff enough as compared with the cantilever. Therefore, more load is thrown upon the latter than it can support, so that it will fail in gravity and tend to overturn until the increasing arch action brings relief and prevents failure. If in such a case the vertical dam slices are considered as independent of each other, as far as cantilever action is concerned, a partial failure in

* Another proof was given by M. Thierry in *Annales des Ponts et Chaussées*, 1888.

cantilever action would result in the vertical dam slices acting as partly fixed beams, supported by elastic supports (arches). This is indeed the case with most of the existing arch dams of reduced gravity section. A typical example will be considered later.

Curved Dams as Cylinder Hoops.

If in a curved dam of any cross-section there exist no vertical cracks from top to bottom, the dam under pressure will always act as a whole, and not (as is generally assumed in elementary gravity design) as a series of vertical slices. Therefore, the calculations with regard to gravity action in curved dams, cannot be made in the same way as usually is done in the case of straight gravity dams, that is, by investigating a vertical slice 1 ft. thick; but the dam has to be treated as a monolith.

To illustrate this, consider a curved dam across a level valley, with perpendicular side slopes. Even if we neglect any kind of reaction and support at the skewbacks of all except the very lowest arch, it is easy to see that (under the influence of external radial pressure, and if there are no open cracks in the upper parts) the dam can fail in gravity only by producing excessive pressure near the points

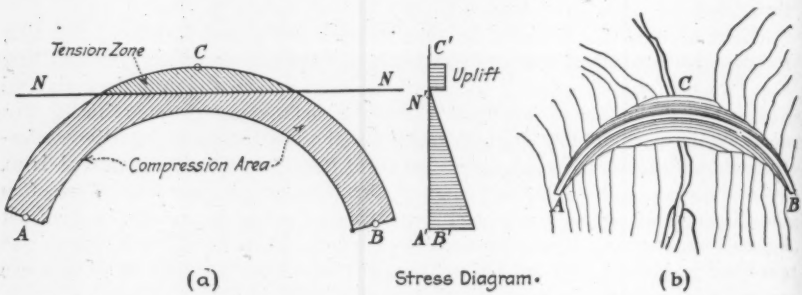

Fig. 11.

A and B, in the plan view, Fig. 11 (a), and tension beyond a certain neutral axis $N\ N$, with a maximum tension near C. Certain internal stresses are also produced which have to be considered separately.

In a good design, the neutral axis $N\ N$ of course should be entirely outside the base by a safe margin. In such a case, no tension occurs between masonry and foundation at any point.

The gravity action of a curved dam, therefore, has to be investigated with due consideration given to these conditions, somewhat as in the design of a brick chimney, so that for safety no tensile stresses can develop at any point of the foundation and so that the compressive stresses are not excessive. This action of a curved dam as a whole is quite clear in such a case as that illustrated above, with the foundation in a horizontal plane and the side abutments in vertical planes; it may be illustrated by imagining a short concrete pipe cut in two lengthwise, and an isolated piece set up vertically. Of course, even without lateral support it will resist external pressure to a much higher degree than would any small vertical slice, if fixed by friction at the bottom only.

As the body of a curved dam resembles somewhat a hollow cylindrical horse-hoof, Fig. 11 (*b*), this kind of gravity action of the entire dam may be called "cylinder-hoof action."

It is important to note that the design of such a dam has to be made in two steps: First, a dam of given or assumed dimensions is investigated with regard to its external stability and to the maximum and minimum stresses upon its foundation at points *A*, *B*, and *C*, respectively, Fig. 11 (*b*). The external forces, total weight of the dam, water pressure and eventually ice pressure, etc., are all combined into one resultant; and the stresses therefrom are determined in a way similar to those in the cases of other eccentrically loaded masonry foundations, or to those of chimneys, bridge piers, etc. The only difference lies in the fact that in the case of curved dams, the reaction of the foundation upon the dam body does not occur upon a horizontal plane (as in the foundations of most other engineering structures) because the area of contact between dam and foundation in general is of varying inclination with the horizontal plane, the foundation of the wings of the dam always lying higher than that of its center part. As an approximation which simplifies very much the determination of the soil pressure, it often may be permitted to substitute for the real and irregular foundation area an inclined plane through the three points *A*, *B*, and *C* of Fig. 11 (*b*).

This part of the investigation of a curved dam refers only to the external stability of the structure, the soil pressure, etc. The internal stresses in a cylinder-hoof dam, due to the water pressure and to the temperature deformations, etc., are of a very complicated nature. In addition to a combination of vertical cantilevers and horizontal arches, there exists in dams of considerable thickness a wedge action in the lower portions where the width of the canyon is small as compared with the thickness of the dam; so that under no circumstances can this part be considered as arching. In the middle and upper portions of the dam where horizontal arch action may occur, the deflection of the cantilever produces torsion in the arch rings, and any horizontal dam section, after the bending of the cantilevers, is warped after deformation. This bending is a maximum in the center of the dam and zero at the side-hills, and these torsion stresses, which hardly can be determined mathematically, tend to stiffen the cantilever and to decrease the deflections of both the cantilever and the arches.

Also, the maximum compressive stresses of the entire structure have a tendency to follow the lines of greatest resistance, which in arch dams in general are not horizontal even in the purely arching parts but are somewhat inclined, with the tendency to form arches of minimum radius and span, between any point in the middle of the dam and the nearest points of the unyielding side abutments.

These torsion and inclined arch stresses, in general, are of too complicated a nature to be determined; also, they are comparatively only small. Both tend to stiffen a curved dam, and may be considered as increasing the safety of the structure. In a practical design, therefore, these complicated stresses may, in general, safely be neglected, and the dam investigated only with regard to combined vertical-cantilever and horizontal-arch action. As already mentioned, the results will be the more accurate the thinner the dam is compared to its height.

If a curved dam, considered as a cylinder-hoof, does not develop between foundation and masonry either excessive compression or any amount of tension

and if a calculation based upon the assumption of combined vertical-cantilever and horizontal-arch action does not indicate in any portion of the dam the existence of stresses which are above a certain allowable limit, then such a curved dam may be considered as a permanently safe structure. In no case will the simple cylinder formula, as generally applied in present practice, give the real stresses in a curved dam; moreover, this formula never should be used as a criterion for the degree of safety of such a structure.

For the determination of the soil pressure of a cylinder-hoof dam, any of the standard methods used in designing masonry foundations may be applied. For all such structures it is assumed that no tension can occur between masonry and

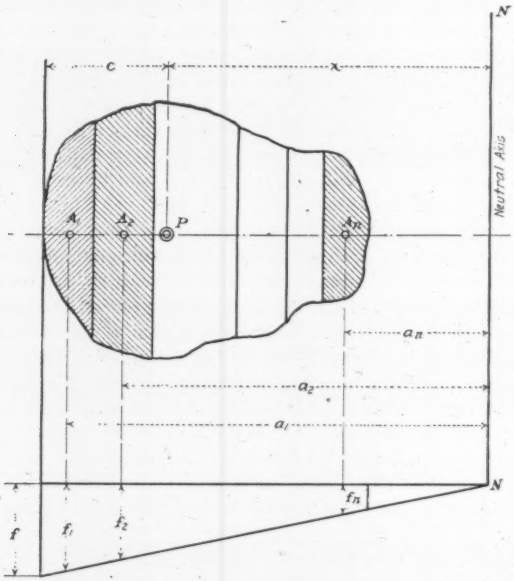

FIG. 12.

ground or bed-rock, and the shape and area of the foundation preferably are so chosen that the whole foundation is under compression. Such is the case if the resultant of all the external forces lies within the "kern" (corresponding to the middle-third of a rectangle) of the foundation area, so that the neutral axis lies outside the area. If the resultant falls outside the kern, the neutral axis divides the foundation area into two parts, of which one is under compression and the other is not stressed at all.

An investigation shows that for a curved dam of full gravity section, the resultant of the external forces lies always within the kern; this is the more certain, the greater the curvature of the dam. The neutral axis therefore always falls outside the foundation area, so that under normal load conditions no tension

occurs at any point of the base. The greater the curvature of such a dam, the farther away is the neutral axis from the foundation area. It is evident that thereby the factor of safety of the dam also is increased correspondingly.

There will now be developed a method by which the foundation stresses in any cylinder-hoof dam of gravity section may be determined. This method is based upon the assumption that the stresses at all points vary in proportion to the distance of these points from the neutral axis. The fundamental condition of the equilibrium of forces, namely, that the total sum of all the elementary soil reactions must equal the resultant of all the external loads, and that this resultant must pass through the center of gravity of the total of the elementary soil reactions, is also applied.

These considerations permit the finding of the locus of the neutral axis, around which the foundation plane is supposed to rotate and produce the soil pressures. The first condition may be written in the form:

$$P = A_1 f_1 + A_2 f_2 + \ldots\ldots\ldots + A_n f_n = \sum_1^n A f \ldots\ldots\ldots (27)$$

if P = resultant of the external forces;

n = number of parts into which the foundation area has been divided (the greater the number, the more accurate are the results);

$A_1 A_2 \ldots A_n$ = the elementary areas;

$f_1 f_2 \ldots f_n$ = the unit stresses upon the areas $A_1 A_2 \ldots A_n$, respectively.

The second condition may be expressed by the equation:

$$A_1 f_1 a_1 + A_2 f_2 a_2 + \ldots\ldots + A_n f_n a_n = \sum_1^n A f a = x P \quad (28)$$

if x = the distance of the resultant and $a_1, a_2 \ldots a_n$ are the distances of the centers of gravity of the elementary areas $A_1 A_2 \ldots A_n$, respectively, from the neutral axis $N N$.

If f is the maximum compression for the points farthest away $(c + x)$ from the neutral axis, then

$$f_1 = f \frac{a_1}{c + x}; \; f_2 = f \frac{a_2}{c + x} \ldots\ldots f_n = f \frac{a_n}{c + x} \ldots\ldots\ldots\ldots (29)$$

Equations (29) introduced in Equations (27) and (28) give

$$A_1 f \frac{a_1}{c + x} + A_2 f \frac{a_2}{c + x} + \ldots\ldots + A_n f \frac{a_n}{c + x} = P \ldots\ldots (27a)$$

$$A_1 f \frac{a_1^2}{c + x} + A_2 f \frac{a_2^2}{c + x} + \ldots\ldots + A_n f \frac{a_n^2}{c + x} = x P \ldots\ldots (28a)$$

From Equation (27a) we obtain

$$f = \frac{P (c + x)}{A_1 a_1 + A_2 a_2 + \ldots + A_n a_n} = \frac{P (c + x)}{\Sigma A a} \ldots\ldots\ldots (27b)$$

and from Equation (28a)

$$f = \frac{x P (c + x)}{A_1 a_1^2 + A_2 a_2^2 + \ldots + A_n a_n^2} = \frac{P x (c + x)}{\Sigma A a^2} \ldots\ldots\ldots (28b)$$

Therefore

$$\frac{P\,(c+x)}{\Sigma\,A\,a} = \frac{P\,x\,(c+x)}{\Sigma\,A\,a^2}$$

and

$$x = \frac{\Sigma\,A\,a^2}{\Sigma\,A\,a}$$

If the elementary areas are very small (n very great), the expression $\Sigma\,A\,a^2$ equals the moment of inertia, I, of the foundation area with regard to the neutral axis $N\,N$. Further, $\Sigma\,A\,a$ is the static moment, S, of the foundation area with regard to $N\,N$, and

$$x = \frac{I}{S} \qquad (30)$$

This formula is correct, only if the neutral axis does not intersect the foundation area, which is the case for all curved dams of gravity section.

As neither the direction nor the locus of the neutral axis is known *à priori*, it is not possible to obtain I and S directly. For a dam with a foundation area which has an axis of symmetry, the neutral axis is perpendicular to this axis of symmetry; and the distance from it may be obtained by a very simple calculation. First, compute the moment of inertia, I_1 for a line 1-1 which is tangent to the up-stream edge of the foundation and parallel with the neutral axis. The moment of inertia I_0 (with regard to a parallel line through the center of gravity, C_g, of the foundation area A) is obtained by the relation:

$$I_0 = I_1 - A\,a_0^2 \qquad (31)^*$$

if a_0 is the distance of C_g from the axis 1-1. On the other hand, the moment of inertia I with regard to the neutral axis is:

$$I = I_0 + A\,(a_0 + e)^2 \qquad (32)$$

if e is the distance between the axis 1-1 and the neutral axis $N\,N$.

From Equations (31) and (32):

$$I = I_1 + 2\,A\,a_0\,e + A\,e^2 \qquad (33)$$

By combining the total weight of the dam body with the total water pressure, the point of intersection P_0 and its distance d from the axis 1-1 may be determined. The distance x of P_0 from the axis $N\,N$ is then $x = d + e$.

The static moment, S_1, of the foundation area with regard to the axis 1-1 may be obtained in any of the well known elementary ways. The static moment S with regard to the neutral axis $N\,N$, then is obtained from the equation

$$S = S_1 + A\,e \qquad (34)$$

Equations (33) and (34) introduced in Equation (30) give:

$$x = d + e = \frac{I}{S} = \frac{I_1 + 2\,A\,a_0\,e + A\,e^2}{S_1 + A\,e}$$

From this equation, the distance e of the neutral axis from the axis 1-1 is:

$$e = \frac{I_1 - d\,S_1}{S_1 + A\,(d - 2\,a_0)} \qquad (35)$$

* "American Civil Engineers' Pocket Book", Mansfield Merriman, Third edition, p. 1190.

The distance $x = d + e$ of the neutral axis from the resultant of the external forces is therefore known; and from Equations (27*b*) and (28*b*) the value for the maximum soil pressure is:

$$f = \frac{P\,(c + x)}{S} = \frac{P\,x\,(c + x)}{I} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

It is of importance to note that this method of determining the foundation stresses is correct only if, as mentioned previously, the entire foundation area is under compression and also if the stresses vary proportionally to their distances from the neutral axis. The direction (not location) of the neutral axis must be known *à priori;* this, in general, is knowable only in the case of symmetrical areas.

If in any foundation, under the assumed load conditions, the value of e as calculated from Equation (35) is of negative sign, this indicates that the neutral axis intersects the foundation area. In such a case, the graphical method as given by O. Mohr* can lead to a correct solution of the problem.

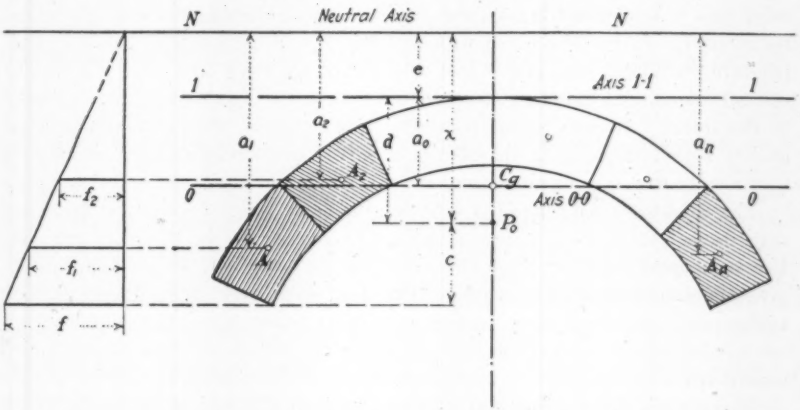

FIG. 13.

In the particular case of curved dams, it is questionable if the stresses over the whole foundation area really are proportional to their distances from the neutral axis, and only in such case would the extreme soil pressure occur at the wings of such a dam. It is entirely possible that a slight bending of the dam wings occurs under the influence of the soil reactions, which bending tends to reduce the maximum stresses near the wings and to transfer them toward the center of the dam. Such a tendency may be accentuated also by temperature and shrinkage deformations. Whatever the influence of such secondary actions may be, it is evident that they will increase the safety of the structure, so that the foregoing calculations will give conservative results.

The theory as outlined applies directly to hoof-dams which have their foundations in a horizontal plane; for example, to a long dam across a level valley, if such a dam consists of a series of hoof-dams of short spans. In the case of single arch dams across narrow canyons, the foundations near the wings lie at higher

* "Abhandlungen aus dem Gebiete der Technischen Mechanik." 1906, p. 262.

elevations than those near the center portions. In such a case, the foregoing method has to be applied with due consideration to the local conditions, also the possibility of vertical arching may have to be considered.

An investigation of the influence of curvature upon the stability of gravity dams shows that the factors of safety against sliding, against excessive soil pressures, against "overturning," etc., are increased very considerably, even in the case of comparatively little curvature. For other than purely statical reasons, any high masonry dam can be built safely only upon a solid rock foundation. Therefore, it is a matter of fact that no curved dam upon such a foundation and with similar side abutments possibly can fail due to excessive soil pressure. Also, it will not even be necessary, in general, to determine the exact value of the foundation stresses, as the compressive strength of solid bed-rock, always will be greater by a safe margin than the strength of the masonry of the dam body. In other words, the external stability of any curved dam of gravity section which is founded upon solid rock is established beyond any doubt. The minimum degree of curvature, therefore, has to be determined with due regard to local conditions and to temperature, shrinkage, etc., and it will be different for every dam site. Hence, the statical calculations of such a structure are to be made in general only with regard to the internal stresses in the masonry. Such stresses may result either from gravity action alone, as in the case of a curved dam of full gravity section, or they may be due to a combination of gravity action and horizontal arching.

The conditions are somewhat different for dams which are long as compared with the height, and which for one reason or another are built straight in plan. For such dams (and for these only) the usual calculations for gravity sections give approximately correct results. On the other hand, such structures have rather small factors of safety, so that their construction either necessitates a very heavy cross-section or involves more or less risk under other than ordinary load conditions.

It is evident that the factor of safety of a gravity dam is increased considerably if the dam is built curved in plan, either in the form of a single arch or of a series of arches, of which each one is of only comparatively short span. Such individual hoof-dams of full gravity section do not require any extra buttresses between the arches, as very little or no horizontal arching occurs in such structures. As a matter of fact, any individual hoof-dam could stand safely, even if for any reason one or both of the adjacent arches should collapse.

Statical Conditions of the Present Types of Curved Dams.

1.—Curved Dams of Gravity Section.

The assertion was made in the foregoing that curved dams of gravity section cannot be depended upon, under normal load conditions, to act also by horizontal arching. This may be proved by the following typical example.

One of the best known curved dams of gravity section (and for which calculations have also been made to determine the amount of horizontal arch action) is the

Lake Cheesman Dam in Colorado.* The principal dimensions of this dam are as follows:

Height,	$H = 227$ ft.;
Thickness,	$t = 176$ ft. at bottom, 18 ft. at crest;
Length at crest,	$L = 700$ ft.;
Radius,	$R = 400$ ft.

The dam is built of granite rubble masonry. Assume $E = 3\,000\,000$ lb. per sq. in. and $c = 0.000\,005$. Further, assume that the temperature of the highest arch slice drops 15° below the closing temperature of the dam. The arch has a tendency to deflect in a down-stream direction for, by Equation (13*b*):

$$D_t = 1.56 \times 400 \times 0.000\,005 \times 15 = 0.047 \text{ ft.}$$

The maximum deflection of the cantilever (triangular form), under full water-pressure, would be approximately, by Equation (4*a*):

$$y_{max.} = \frac{62.5 \times 227^5}{3\,000\,000 \times 144 \times 176^3} = 0.016 \text{ ft.}$$

The maximum deflection of the cantilever under full water pressure is only about one-third as much as the deflection of the highest arches due to temperature alone. This means that at a time when the arch temperature is 5° or more below normal, the cantilever will not be supported at all by horizontal arch action, and therefore will have to support the entire water pressure. The conclusion reached by Mr. Woodard (that the upper horizontal arches support nearly half of the load at the top,† etc.) therefore may be correct only for the very special case that the length of the arches has not changed either by temperature or by shrinkage from the theoretical length at the time of closing. On the other hand, at a time of high dam temperature, horizontal arching may exist to a very considerable degree, but this condition of course will not give to the dam any higher factor of safety and at certain times it may even produce reversed stresses in the cantilever.

2.—Dams of Reduced Gravity Section.

It can be proved that all arch dams of reduced gravity section for which it was assumed that considerable horizontal arching exists by abutting against the side-hills either have failed partly in cantilever action or they are depending for their stability almost entirely upon their form as cylinder-hoofs. A typical example of this is the famous Zola Dam in France.

It can be shown that, in this dam, at times of low temperature, the horizontal arches have a tendency to deflect more than the cantilever can elastically, in other words, at certain times the cantilever is not supported by the arches. On the other hand, the cantilever is too slender to support alone the entire water pressure, if the stresses are calculated by the usual methods of gravity dam design.

The Zola Dam is about 120 ft. high, 41.8 ft. thick at the base, and 19 ft. thick at the crest. The radius is 158 ft. and the length at the crest is 205 ft. The dam is built of uncoursed rubble masonry of a very good quality, for which $E = 3\,500\,000$ lb. per sq. in and $c = 0.000\,005$, say. A drop of 25° Fahr. in the

* *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 89.
† *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 125.

temperature of the highest arches tends to produce a deflection, by Equation (13*b*), of

$$D_t = 1.56 \times 158 \times 0.000\,005 \times 25 = 0.032 \text{ ft.}$$

The maximum deflection of the highest vertical cantilever under full water pressure, assuming first the dam to be of a constant thickness, 41.8 ft., from top to bottom, by Equation (4*c*), would be

$$D' = \frac{62.5 \times 120^5}{2.5 \times 3\,500\,000 \times 144 \times 41.8^3} = 0.017 \text{ ft.}$$

If the cross-section of the dam were of a triangular form instead (top width zero), the deflection of the cantilever then would be

$$D'' = 2.5\ D' = 0.042 \text{ ft.}$$

The deflection of the cantilever under total load would be somewhere between the two values D' and D'' and probably less than the deflection of the upper arch due to a fall in temperature alone. The assertion made previously that at the time of low dam temperature the cantilever is not supported by horizontal arching, as generally understood, is thus verified. An investigation of a section somewhat nearer the sides or abutments, where the cantilever is shorter and much stiffer, shows the conditions still more clearly.

One of the most notable curved dams in America is the old Bear Valley Dam in California. This structure is curved in plan, with a radius of 335 ft. The height above a substructure which in dimensions approaches a gravity section, is 48 ft. The dam has a thickness of 3.2 ft. at the crest, and of 8.4 ft. only at the depth of 48 ft. It is built of uncoursed rubble granite masonry of a very good quality.

Assuming $E = 3\,500\,000$ lb. per sq. in., and $c = 0.000\,005$, the deflection of the highest arch due to a decrease in temperature of 25° Fahr., by Equation (13*b*), is:

$$D_t = 1.56 \times 335 \times 0.000\,005 \times 25 = 0.065 \text{ ft.}$$

Equation (4*a*) gives for the maximum deflection of the cantilever assumed of triangular form, under full water pressure:

$$D' = \frac{62.5 \times 48^5}{3\,500\,000 \times 144 \times 8.4^3} = 0.054 \text{ ft.}$$

Also, in this case the cantilever at the time of low temperature would not be supported by horizontal arching.

The Bear Valley Dam was always considered a rather unsafe structure, and with its slender cross-section and slight curvature, it exceeds in boldness any of its kind. The above calculations show clearly that the cantilever must have been cracked when the water rose to the crest in the first winter. Nevertheless, the horizontal arch action which could develop after the cantilever had deflected accordingly, was able easily to prevent a collapse, as the actual stresses in the dam body still were considerably below the crushing strength of the masonry. This is the only possible explanation for the long and successful service of this most famous among all curved dams. At the same time, this structure has proved the great possibilities of the arched form of masonry dams, so that, with certain

improvements, structures similar to the Bear Valley Dam may be built with safety in the future.

3.—Constant-Angle Arch Dams.

A representative example of this type is the Lake Spaulding Dam* in California, built in 1912-13.

This dam was designed first as a curved gravity dam of 260-ft. height. The lower portion, about 60 ft., had been built when the design was changed and the principle of the constant-angle arch dam applied. This resulted in a material reduction of the cross-section, and the dam was built thus for an additional 165 ft. of height. The maximum stresses in the arches of the completed dam were figured as being somewhat less than 350 lb. per sq. ft., under the assumption that the horizontal arches supported all the water pressure. Apparently, no temperature or shrinkage stresses, etc., were considered.

As a matter of fact, such neglected stresses must have been of considerable magnitude. L. R. Jorgensen, M. Am. Soc. C. E., says in his description of this dam:†

"During the latter part of 1913 there was very little water in the reservoir, the outside temperature was low, and the chemical heat was practically out of the dam body. This had the effect of opening up contraction joints (which are 80 ft. apart) about $\frac{1}{8}$ in. In addition to this, cracks about $\frac{1}{16}$ in. wide, appeared midway between each contraction joint. These were all along radial lines, the same as the contraction joints. When the water in the reservoir rose to the crest, early in February, 1914, these cracks closed."

The closing of the cracks in a comparatively short time, with the average dam temperature probably not changed very much, was due mostly to the deflection of arches and cantilever in a down-stream direction. These deflections were measured at different times (for instance, on February 10th, 1914, at the center of the dam crest) and, with the water 5 ft. below the crest, were found to be 3.44 in.,‡ measured from a zero mark set November 25th, 1913.

The radius of the highest arch§ (at Elevation 4 825) was about 390 ft., its length 600 ft., the rise of the arc 105 ft., and the crest width 20 ft. To close the visible cracks (which were about $\frac{3}{16}$ in. per 80 ft., or 1.4 in. = 0.117 ft. in the 600 ft. of total length), the highest arch, Equation (11), must have had to be deflected:

$$D = \frac{3}{16} \times \frac{600}{105} \times 0.117 = 0.125 \text{ ft.} = 1.50 \text{ in.}$$

The greater shortening of the arch (resulting in a total deflection of 3.44 in. as measured) was due to relieving internal tension stresses, and of course also to actual compression.

To investigate the action of the vertical cantilever on the top of which the deflections were measured, assume first that the cantilever carried all the load, which it doubtless had to do, at least before the vertical cracks were closed and before horizontal arching could develop. By Equation (4*a*) this deflection is

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 710.

† *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 715.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 730.

§ The dam was raised in 1916 and again in 1919. The present investigations are made only for the dam as it existed before being raised.

obtained, under the assumption that the thickness of the dam decreases uniformly from the maximum at the bottom to zero at the crest.

The conditions at the Lake Spaulding Dam are somewhat unusual owing to the fact that its lowest 60 ft. are of gravity section, and that only from that point up is the dam designed as depending on horizontal arching. Therefore, assume that the cantilever starts at the top of the gravity section, having thus a height of 165 ft. with a maximum thickness at its base of about 96 ft.

For a triangular cantilever of these dimensions and assuming the water risen to the crest of the dam, and $E = 2\,000\,000$ lb. per sq. in., Equation (4*a*) would give the maximum deflection of the crest as follows:

$$D = \frac{62.5 \times 165^5}{2\,000\,000 \times 144 \times 96^3} = 0.030 \text{ ft.} = 0.36 \text{ in.}$$

This value (compared with the deflection as actually measured, or 3.44 in.) shows that the cantilever deflected, as measured, about ten times as much as the laws of elasticity really would permit. This is possible only if the cantilever broke; and it appears there must have opened on the water side of the dam horizontal cracks of considerable size, aggregating probably not less than $\frac{1}{2}$ in. at the up-stream face. After the cantilever had been broken, arch action could develop and carry the water pressure.

It may now be of interest to investigate the dam with regard to the temperature stresses in the arching parts.

At Elevation 4 750 (that is, 75 ft. below the crest of the dam) the thickness t is about 50 ft. and the radius R of the center line equals 345 ft. Assuming a maximum decrease of temperature of 25° Fahr. below normal at the crest of the dam, and of zero at the bottom of the assumed cantilever, the change of temperature of the arch slice under investigation is:

$$T = 25 \times \frac{90}{165} = 13.7° \text{ Fahr.}$$

By Equation (17) this produces in a horizontal arch slice 1 ft. thick a negative thrust as follows:

$$H_t = -0.94 \frac{50^3}{65^2} 2\,000\,000 \times 144 \times 0.000\,006 \times 13.7 = -650\,000 \text{ lb.}$$

where $c = 0.000\,006$ and the rise h of the arch slice has been taken at 65 ft.

The moment at the arch crown (considering that H_t acts at a distance of $\frac{h}{3}$ from it), has the value

$$M = -650\,000 \times \frac{65}{3} = -14\,000\,000 \text{ ft-lb.}$$

The resulting stresses are

$$f = \frac{H_t}{t} \pm \frac{M}{\text{Sec. Mod.}} = \frac{-650\,000}{50} \pm \frac{14\,000\,000}{\frac{1}{6}\,50^2} = -13\,000 \pm 33\,600$$

$f_1 = -46\,000$ lb. per sq. ft. (tension, intrados).
$f_2 = +20\,600$ lb. per sq. ft. (compression, extrados).

The stresses due to shrinkage of the concrete during setting and hardening (and also those due to the shortening of the arches under water pressure) may be investigated in the same manner. Even under conservative assumptions, they would be found to be of such magnitudes that the actual cracking of the dam along vertical planes, due to all the stresses combined, is readily explained.

These investigations prove beyond doubt that this dam must have developed cracks not only in vertical, but also in horizontal, planes. It seems, therefore, that some engineers have come in dam design to a rather dangerous point, where it is no longer safe to leave certain stresses simply to the so-called "big factor of safety" to take care of. The Lake Spaulding Dam has been under pressure for a number of years, and has even been raised twice. Nevertheless, these conditions hardly can be called satisfactory; at least not from the point of view of scientific dam design.

In Europe very few curved dams of reduced gravity section have been built. Nevertheless, most gravity dams are built curved in plan, but admittedly* only for the reason that temperature and shrinkage cracks thus tend to be closed when the dam is under pressure. This reason alone hardly would justify the increased expenditures due to the greater volume of a curved gravity dam over a straight one, as the curvature always has to be considerable to obtain those desired results. It seems that it was more some kind of engineering instinct than real knowledge of the actual conditions (cylinder-hoof action, etc.) which brought the curved type of gravity dams into favor.

The unit stresses in all European dams always have been assumed rather low, and only the American engineers of the West have gone in their assumptions as high as 350 lb. per sq. in. It is often stated in published reports of arch dams that the maximum unit stresses, even if the arches are assumed to carry all the load, are less than 200 to 300 lb. per sq. in. The factor of safety of the structure then is assumed to be correspondingly large, notwithstanding the fact that many of those structures have developed cracks; or, in plain words, have developed tensile stresses of nearly as high values as the so-called maximum compressive stresses. As the temperature in some cases may rise above normal as much as it fell below when producing cracks, the additional compressive stresses due to a rise of temperature also certainly will be considerable. Each degree change in temperature, Fahrenheit, in restrained concrete produces stresses of 10 to 15 lb. per sq. in. The before-mentioned casual claims with regard to low maximum unit stresses and correspondingly high factors of safety, therefore, are entirely erroneous and misleading, so far as arch dams are concerned.

As a matter of fact, hardly any curved masonry dams exist, which depend for their stability on horizontal arch action, and for which the average dam temperature changes about $\pm$ 20 to 25° Fahr., that are not subject either to excessive tensile stresses or to compressive stresses of not less than 500 lb. per sq. in., or to both. Experience has shown that the material used in building these dams can withstand such stresses, as no curved dam has collapsed as yet. As a consequence, it follows that (if the maximum stresses in a dam can be determined more or less accurately and if the structure is designed accordingly) there is no reason why the unit stresses generally assumed in dam design should be much

* A. Ludin, "Die Wasserkräfte", Berlin, 1913, Vol. II, p. 959.

lower than in other concrete structures. Even a 1:3:5 concrete (after 3 to 6 months, that is, when the dam may come under pressure) has an ultimate strength of hardly less than 2 000 lb. per sq. in. A factor of safety of about 4 with respect to the crushing strength and of 2 to 3 with respect to the elastic limit of the concrete should be sufficient, when the real maximum stresses according to a scientific design are considered, and when the structure is so designed that at no place tension develops unless it is taken care of by sufficient metal reinforcement.

Such assumptions would mean allowable unit stresses in compression of not less than 500 lb. per sq. in. This may seem extreme to many engineers, but an investigation of existing arch dams shows that such pressures are most probable for all of them, and that they are a certainty for all arch dams of reduced gravity section which have opened so-called "unavoidable expansion cracks."

It can be proved that in fact it is impossible to construct an arch dam of reduced gravity section which does not develop considerable tension and probably also open horizontal cracks in the vertical cantilever, unless the masonry is reinforced by steel bars. A change in the dam temperature of 10 to 15° Fahr. is sufficient in most cases to produce arch deflections which the cantilever under water pressure is not able to follow elastically. The reason for this is that such a decrease in temperature tends to deflect the horizontal arches so much that they cannot give enough support to the vertical cantilever, so that the latter has to carry all, or nearly all, the pressure of the rising water. No dam (except one of gravity section) can do this without producing cracks, unless it is reinforced sufficiently in a vertical direction. This reinforcement of course has to be well anchored into the solid foundation rock, and has to extend into the dam body as far upward as tension is likely to occur. As far as the writer knows, no dam built thus far has been provided with sufficient reinforcement, if any, to take safely these tensile stresses; and as a consequence, all existing curved dams of reduced gravity section must have developed cracks in the zones of excessive tension. This is not so alarming as it may seem at first thought, because the maximum compressive stresses in general are still considerably below the crushing strength of the masonry, and such a dam therefore still may have a sufficient factor of safety to prevent a collapse under ordinary load and temperature conditions. But a dangerous point is reached if a dam, in addition to excessive tension in the cantilever part, also has acquired vertical cracks due to temperature changes, shrinkage, etc. Such a dam will consist of more or less loose blocks of masonry, similar to huge radial bricks, for which friction between the surfaces is the principal means of resistance against the water pressure.

It remains to be seen whether certain high arch dams which have developed cracks soon after construction will stand reversible stresses for an indefinite period.

Fortunately, there is one thing worthy of mention which relieves automatically (at least in part) internal stresses and extreme stresses and which probably has saved many of our masonry structures from certain failure. Concrete obtains the maximum degree of hardness only after years, and hence retains during all this

time a certain degree of plasticity,* which in a stressed structure has the tendency to equalize the extremes of the stresses. This fact is of great importance in all statically indeterminate structures, and particularly so in the case of curved concrete dams; but it hardly can be called good practice to rely too much on this feature.

With regard to the expression "factor of safety" of an arch dam, it is difficult to say what is really meant by it. If some engineers have claimed the factor of safety of their dams to be 5 to 6, etc., they hardly meant to say that the structure would fail only when the dam was overtopped by twice or three times the assumed maximum depth of the water. It should also be considered that when a storage dam under extraordinary conditions really becomes subject to a considerable overflow, the originally calculated distribution of load between cantilever and arches (taken in percentages) changes somewhat, so that in such a case the maximum stresses in the dam do increase more than proportionately to the increase in the horizontal pressure.

If a dam is considered as subject to the pressure of a liquid of greater specific gravity than water, the stresses in the dam due to this pressure will increase nearly in direct proportion to the specific gravity of the new liquid. The writer would suggest as the definition of the factor of safety of a dam the value of the specific gravity of a liquid which, replacing the water in the reservoir and under extreme temperature conditions, ice pressure, etc., would produce failure of the structure.

It is often claimed that structures of the importance of high storage dams ought to be built with exceptionally large factors of safety. This sounds reasonable, on account of the danger of great losses in lives and property in case of a collapse; but in reality, the conditions are very different. The whole of Egypt lies quietly at the foot of the Assuan Dam, which, as a dam of pure gravity section, has a factor of safety probably less than 2. For arch dams factors of safety of 5 or 6 have been required, but people do not mind sleeping in the shadow of such structures which have cracked already under excessive stresses. These considerations show that both the public and the majority among engineers necessarily will have to modify their opinions with regard to the real safety of many of the existing masonry dams. The conclusions for future construction are self-evident.

General Discussion.

The methods outlined in this paper will provide means for designing dams more scientifically than has been customary. It is to be hoped that the graphical method of design here given will be used in the future in preference to the cylinder formula. This will increase the safety and at the same time decrease materially the cost of such dams.

Most of the formulas developed here, for the sake of simplicity, are only approximations, but nevertheless are within a small percentage of the theoretically correct values. They are sufficiently close for most cases, as the other assumptions in design (weight per cubic foot of masonry, modulus of elasticity, changes of

* A. H. Fuller and C. C. Moore, "Tests Showing Continued Deformation under Constant Load"; C. B. Smith, "The Flow of Concrete under Restrained Load." These two papers were read at the Twelfth Annual Convention of the American Concrete Institute, in Chicago. See also abstracts in *Engineering Record*, March, 1916; also "The University of Minnesota Studies in Engineering, No. 3", by F. R. McMillan.

temperature, linear coefficient of expansion, degree of shrinkage, and, last but not least, the statically very uncertain conditions of the arch abutments) in general would involve much greater uncertainties, so that more accurate calculations probably would be only unnecessary refinements, without leading to an appreciably higher factor of safety or to economy in material.

It is to be hoped that the better knowledge of the conditions as they exist in solid masonry dams, especially with respect to cylinder-hoof and to combined cantilever and arch action, will result in new developments and improvements in the design and construction of such dams.

There is no doubt that it will be practicable to create a new type of curved dam, in which both cantilever and arch action will combine harmoniously. Such a structure will be more economical and generally safer than either a gravity or a theoretically pure arch dam. From the standpoint of economy, it will be of great advantage that the forces acting in such a structure can be determined fairly accurately, as by decreasing the factor of ignorance in the design it will be allowable to use higher working stresses and consequently smaller cross-sections. Material and workmanship of course will have to be correspondingly better.

A very important consideration is the cylinder-hoof action, especially with respect to the external stability of arch dams, for which the statical calculations are already known in principle to every engineer, thus permitting the determination of the foundation stresses in a very elementary manner.

In the case of a curved dam of full gravity section, the internal stresses may be considered as due entirely to vertical cantilever action, as no horizontal arching can occur at the time of low temperature. For every isolated vertical slice of a straight gravity dam, there exists a factor of safety of less than 2 with respect to the external stability (sliding, excessive soil pressure, etc.). The factor of safety of such a gravity dam may easily be doubled, or more, by building the dam curved in plan. In addition to this, if we were able to determine the torsion and inclined arch action in curved dams, it would be practicable to construct such dams of somewhat reduced gravity section, which, nevertheless, would have a considerably greater factor of safety than straight dams of standard gravity profile.

Without going into any details here it is easy to see that (as far as curved gravity dams in particular are concerned) the entire "middle-third theory" and many of the more complicated formulas and calculations (with respect to the most economical top width, to the varying batters of their up-stream and down-stream faces, etc.) are only very unnecessary refinements; they should be modified, if such dams are calculated (as they ought to be) as cylinder-hoofs.

The object of the investigation here presented is principally to call attention to the many wrong and unwarranted assumptions of the usually applied methods of dam design, and to indicate ways and means of improving the present methods of design. It was found impracticable to extend this paper more than to give a general outline of the new theory of curved dams. Nevertheless, it will be practicable for every engineer to apply these principles, and thus obtain a much better knowledge of the real strengths and factors of safety of existing dams. It will be found that, for new designs, in most cases considerable economy in material also may be effected; but to do this will require a somewhat different distribution of

the material by strengthening weak points and by saving material in places where it would be pure waste, or even worse.

All the investigations here presented and the conclusions reached, apply (with logical modifications) to all and any kind of curved dams, low or high, with or without overflow, and of single or multiple arch types. The conditions at various dam sites are so different in many respects that the theories here given of course have to be applied with due regard to local circumstances. They also will be of help in choosing the most economical type of dam for a given site.

Conclusions.

1.—A long straight dam of standard gravity section, in general, has a real factor of safety of hardly more than 1.5. The stability is increased considerably by making such a dam curved in an up-stream direction, for very long dams, eventually in the form of a series of arches of 300 to 400 ft. span. The increase in safety, due to the curved form, is not primarily due to horizontal arching, as often has been stated, but instead is the result of cylinder-hoof action, which gives a curved dam a much greater degree of stability than is possessed by a straight dam of the same cross-section. The factor of safety of a straight gravity dam is so small, and the increase in stability due to cylinder-hoof action is so great even in the case of comparatively small curvature, that no solid masonry dam should be built straight in plan.

2.—The design of curved dams in which little or no horizontal arching can occur ought to be based entirely upon the theory of cylinder-hoof action. The factor of safety of such a structure, even with somewhat reduced gravity section, may be made considerably higher than in a straight dam of full gravity section. It can be shown by theory (and many examples of existing dams prove it) that in this way a considerable economy of material also may be obtained.

3.—It is practicable to determine for every arch dam approximately what percentage of the water pressure is supported respectively by cantilever and by horizontal arch action.

4.—Curved dams of gravity section cannot be depended upon to have any appreciable amount of water pressure supported by horizontal arch action. Arching nevertheless may prevent ultimate failure in cases where the cantilever action has been destroyed partly, or where the dam has begun to slide.

5.—Every curved dam, except eventually one of gravity section, should be designed with due consideration of the effects of shrinkage and of changes of temperature. This has to be done not only for the calculation of the load distribution between cantilevers and horizontal arches, but also because of the stresses due to bending moments. For average conditions it may be assumed that the maximum change of temperature in a dam body is $\pm$ 25° Fahr. at the crest of the dam and diminishes at a uniform rate to nothing at the bottom.

6.—In a dam which depends for its stability to a considerable degree upon horizontal arch action, the occurrence of vertical cracks ought to be prevented by all practicable means. The influence of the shrinkage of the concrete should be investigated very carefully and means provided to prevent the opening of cracks. For practical calculations it may be assumed that the effect of shrinkage is zero at the bottom of the dam, and increases at a uniform rate to a maximum

at the top. This maximum depends largely upon the composition of the concrete, the methods of placing the same, and the arrangement of the construction joints. The effect of shrinkage is similar to that of a decrease in temperature. For concrete, this shrinkage never should be assumed to be less than that due to a fall of 15° to 25° Fahr. No shrinkage has to be considered for a dam of well coursed masonry.

7.—The design of a dam which crosses a narrow canyon should be made with due consideration to the possibility of vertical arching.

8.—The calculations of the stresses in arch dams necessarily have to be based upon certain assumptions, whatever the method adopted for the design of such dams may be. The most prominent engineers disagree somewhat among themselves, for instance, as to the modulus of elasticity, the degree of shrinkage, and the range of temperature, etc., in slender arch dams. It is to be hoped that these points will be discussed fully by all those who may be in possession of useful information bearing on this subject. Further, it would be most desirable if for existing arch dams, as far as practicable, temperature and deflection measurements, combined eventually with direct stress measurements, were made periodically, and published.

It is of the greatest importance that such measurements be made for all arch dams built in the future. In fact, the State authorities which have to pass on the design of all dams ought to require that adequate means be provided to measure temperature stresses and deflections for arch dams during construction as well as for a period of several years after the dam is completed. Such continuous records would be most valuable, not only in connection with the dam for which they were made, but also in the design of future similar structures.

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DISCUSSION

F. J. NEUBAUER,* ESQ. (by letter).—The writer was much interested in the paper by B. A. Smith, M. Am. Soc. C. E., on "Arched Dams,"† and in the author's analysis of the same problem. It is of special interest to note how these two authors attack this problem; Mr. Smith wholly by mathematical theory, and Mr. Noetzli by a combined theoretical and graphical method. Much may be said in favor of both methods. The writer's interest lies chiefly in the different ways in which the two engineers use the tool, mathematics.

That the subject of the design of arch dams is not yet exhausted is due simply to the fact that the apparent discrepancy between the practical experience with them on the one hand and applied theory in their design on the other, is too great to be easily accounted for. Each new paper on the subject seems to try to overcome the difficulty by introducing new or different mathematical formulas. The writer believes the discrepancy is due merely to differences in the interpretations of the mathematical formulas.

Mathematics is merely a short-hand method of long-hand thinking, and should be used in that way when applied to practical problems. Unfortunately, the young engineer just out of college is not, as a rule, of sufficient mental (or rather, mathematical) maturity to comprehend fully the usefulness of mathematics as a working tool, and the older engineer often is too busy otherwise to give more than a little attention to the mathematics, and relies considerably on his handbook.

Both papers have this good feature—they will send the inquisitive back to the dust-covered calculus, and perhaps cause them to ask "What are Bessel's functions"?

Mr. Noetzli's paper is preferable in the sense that the average engineer can follow it understandingly. The mathematics involved is not beyond what many can remember from their college days, and the faithful handbook will supply the rest. Viewed from a purely engineering standpoint, there are many good points which will be more emphasized in the discussions; but there are some weak points which each individual design will bring out.

Mr. Noetzli seems to indicate that the computation involved by Mr. Smith's method of design is too laborious. The writer does not agree with him in this; as a matter of fact, the computation involved in Mr. Smith's method is just as simple as that in Mr. Noetzli's when one has a working knowledge of Bessel's or Michell's functions.

Mr. Smith's paper is a brilliant one. It gives us not only something new, but does so in a complete form. As the problem with its solution is given by him, one can follow the entire discussion step by step without once losing the point the author wishes to bring out. It is a good example of what a textbook on applied mathematics should be. The references he gives (Gray and Mathews' "Treatise on Bessel Functions"), however, is, the writer understands, out of print, and, consequently, rather hard to obtain by one who wishes to familiarize himself with the mathematics as used by Mr. Smith. However, Byerly's "Fourier Series and Spherical Harmonics" may be had in any good library. Because of the way that

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† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

Mr. Smith has given the solution, however, one does not need any other reference, except perhaps to some elementary introduction to differential equations, as given in most texts on calculus.

The writer is not as yet familiar with the so-called Michell functions, but sees the great advantage of introducing them in this case. The solution of Mr. Smith's differential Equation (37):

$$\frac{d^2}{d\,x^2}\left(x^3 \times \frac{d^2\,u}{d\,x^2}\right) + x\,u = 0$$

is noteworthy.

This is the first time the writer has seen "Cylindrical Harmonics" applied to a practical problem in civil engineering. Of course, they have been used for nearly one hundred years in problems of heat, electricity, and geodesy, and especially in astronomy. Bessel used them first in astronomy, although Fourier was their originator.

When one stops to consider, the applied mathematics used by Mr. Smith is the most logical method for the design of cylindrical dams. He ought to be congratulated for giving the teacher of applied mathematics a really good practical problem. The writer hopes he will present more problems of that type. The apparent disadvantage of his paper is, of course, the mathematics involved. Few engineers go beyond the calculus, for no other reason than that they are not required to study any higher mathematics, or made to see the usefulness thereof; but a simple text like Perry's "Calculus for Engineers" brings out all the mathematics that are involved, including "Zonal Harmonics" and Bessel's functions.

The writer firmly believes that papers like these by Messrs. Smith and Noetzli are valuable in indicating how applied mathematics should be taught. A few such papers, discussed intelligently in a classroom with young engineers, will do much more good than merely following an adopted text from page to page, and wasting time in solving useless problems. Unfortunately, many teachers of mathematics in engineering courses are teachers of mathematics first, and only secondarily teachers of engineering mathematics.

The writer wishes to bring out a point, common to all such problems as are presented in these two papers. In each paper the author verifies his theory by a practical case, and thus convinces the readers of the correctness of his ideas. To verify a theory by something already designed is always different from applying the theory to a new case. There is no such ideal case as "average", each individual new design demands a new application of the theory, differing slightly from the original application. Here is where the engineer has that powerful reserve, common sense, and it is not the fault of the theory or of the mathematics involved that causes the discrepancy, but the manner in which the theory and the mathematics are applied.

ALLEN HAZEN,* M. AM. SOC. C. E. (by letter).—In comparing the relative stability of straight and arch dams, there is one important difference in the manner in which the calculations are made.

* New York City.

In a straight dam, weight is relied on for stability. Weight can be estimated accurately; and when so estimated, one can be sure that it is all there, but it is also quite certain that there is no additional amount in reserve for contingencies.

With an arch dam, on the other hand, the crushing strength controls. It is usual to take values for crushing strength so low that with the hardest rocks and best masonry it is certain that the actual strengths are many times the values used in calculation. There is thus a reserve element in stability, which has no counterpart in a gravity dam. Even though under extreme conditions there should be sliding, cracking, and spalling off of the corners, it is certain that the arch will come to a solid bearing that will permit it to withstand very great pressure.

Experience with experimental arches, tested to destruction, as well as with some arch dams in which there are crushing stresses several times greater than those commonly used, may be taken as ample evidence of this condition.

One of the surprising practical experiences is that even in those cases where very high unit stresses are computed, and where these stresses certainly exist, there is absence of evidences of such moving, cracking, and spalling as might be expected from theoretical considerations. The number of arch dams which have been constructed is very considerable. In some cases, very high stresses have been used; but, nevertheless, there has been a conspicuous absence of failures.

The paper under discussion, and also that of B. A. Smith, M. Am. Soc. C. E., on "Arched Dams",* may be taken as contributions to this subject, which is not a new one. A report on the Quaker Bridge Dam,† published by the Aqueduct Commission of the City of New York in 1889, may be mentioned as a record of a very earnest study of this problem. It shows that, in 1887, the late A. Fteley, Past-President, Am. Soc. C. E., Consulting Engineer, and the late B. F. Church, M. Am. Soc. C. E., Chief Engineer, reported in favor of a straight dam, and gave their reasons in detail. Mr. Fteley stated (page 42):

"That a dam of such a length as Quaker Bridge Dam, if acting as an arch, would be subjected to excessive strains.

"That, with the profile recommended, it can successfully resist in all its parts, the pressures applied to it, without any additional support.

"That if built on a curve, it would not act as an arch, and would be more expensive.

"That it is advisable that the dam should be built in a straight line."

Afterward, a Board of Experts composed of the late Joseph P. Davis, M. Am. Soc. C. E., J. James R. Croes, Past-President, Am. Soc. C. E., and Mr. William F. Shunk, in 1888, reached the conclusions:

"*First.*—That in designing a dam to close a deep narrow gorge, it is safe to give a curved form in plan and to rely upon arch action for its stability; if the radius is short, the cross-section of the dam may be reduced below what is termed the gravity section, meaning thereby a cross-section or profile of such proportions that it is able by the force of gravity alone to resist the forces tending to overturn it or to slide it on its base at any point.

"*Second.*—That a gravity dam built in plan on a curve of long radius, derives no appreciable aid from arch action, so long as the masonry remains intact; but that in case of a yielding of the masonry the curved form might prove of advantage.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

† On file in the Engineering Societies Library.

"The division between what may be called a long radius and what may be called a short radius, is, of course, indefinite, and depends somewhat upon the height of the dam. In a general way we would speak of a radius under three hundred (300) feet as a short one, and one of over six hundred (600) feet as a long one, for a dam of the height herein contemplated.

"*Third.*—That in a structure of the magnitude and importance of the Quaker Bridge Dam, the question of producing a pleasing architectural effect is second only to that of structural stability, and that such an effect can be better obtained by a plan curved regularly on a long radius than by a plan composed of straight lines with sharp angular deflections.

"*Fourth.*—That the curved form better accommodates itself to changes of volume due to changes of temperature."

Thirty years afterward none of those who took part in the discussion is living, but their record will well repay study by those interested in masonry dam construction.

H. J. KESNER,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper presents a conclusion regarding the proportion of hydrostatic pressure carried by vertical cantilever action and that carried by horizontal arching, which the writer believes to be open to argument. For example, the author finds in the Pathfinder Dam that above Elevation 177 the load carried by horizontal arching is in excess of that due to full hydrostatic pressure, giving a distribution of loads on arch and cantilever elements similar to that indicated in his Fig. 4 (*c*).

It seems evident that if the arch elements above Point *A* of this diagram are carrying a pressure greater than the hydrostatic pressure, the dam as a whole at some section below *A* must carry less than hydrostatic pressure, and that whatever area, *ADD'*, is added to the hydrostatic triangle, *CDB*, above *A*, must be subtracted somewhere below *A*.

The Shoshone Dam has in its upper portion a cross-section almost identical with that of the Pathfinder Dam; but it has a total height of 300 ft. If the author's conclusions are correct, one would expect to find toward the top of the Shoshone Dam a pressure exerted on the horizontal arch elements considerably in excess of hydrostatic pressure.

An analysis of the pressures in the Shoshone Dam appears on a sheet published by the U. S. Reclamation Service in 1917. The results of this same analysis appear on page 474 of the edition of "Irrigation Engineering" by Davis and Wilson, published in 1919. The "water-pressure diagram" there shown exhibits the distribution of pressures between arch and cantilever elements of the dam at all elevations. There is no portion of the dam near the top in which the arch carries more than hydrostatic pressure, as would be anticipated from Mr. Noetzli's conclusions.

In the assumptions underlying the computations for the Shoshone Dam, no considerable departures from those used by Mr. Noetzli are found. The deflection of a cantilever element is computed from a formula identical with his Equation (3), and the deflection of an arch element is computed from a formula similar to his Equation (13), the only difference being in the numerical coefficient. As will be shown, however, this difference in coefficients produces comparatively small changes in the distribution of pressures. Instead of trial

* Lafayette, Ind.

deflection curves which later may be adjusted and fitted together to exhibit the actual deflections of the structure, the Shoshone Dam analysis contains an analytical formula from which it is possible to plot the distribution of pressures at any elevation, thus developing the "water-pressure diagram", Fig. 14. This operation may be accomplished by the formula:

$$K_a = \frac{100\,000\,x^2 - 308\,x^3}{2\,R^2\,(300 - x) - 4.92\,x^3 + 1\,599\,x^2},$$

the notation being as indicated on the diagram.

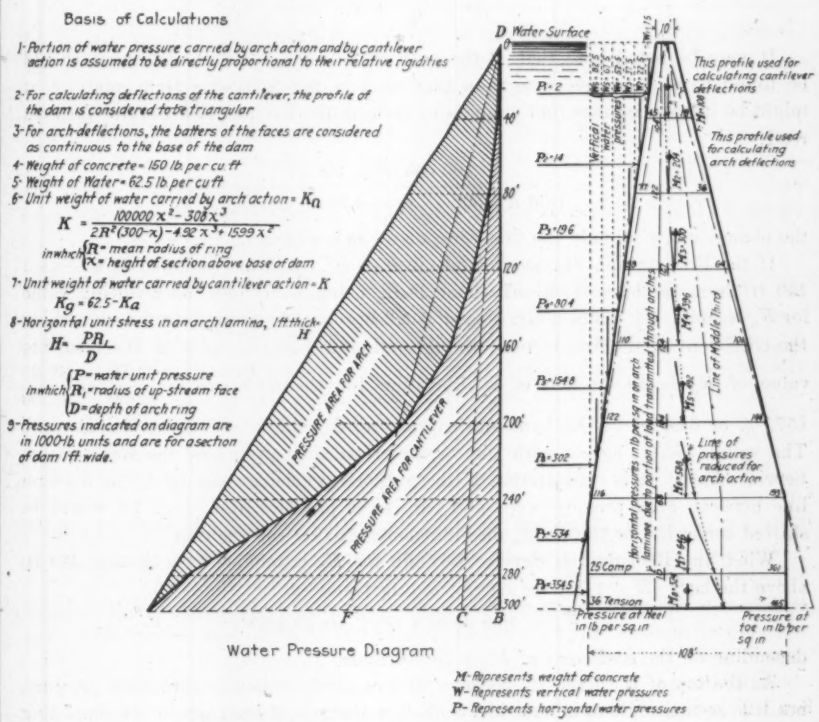


FIG. 14.—SHOSHONE DAM—ANALYSIS OF PRESSURES.

In his discussion on the "Lake Cheesman Dam and Reservoir",* Edwin Duryea, M. Am. Soc. C. E., developed a formula of the form:

$$K_a = \frac{125\,x^2}{r^2 + 2\,x^2}$$

for a triangular dam with a base width of one-half the height. The formula used for the Shoshone Dam analysis may be developed in a manner similar to that published by Mr. Duryea, but adapted to a trapezoidal section. The constants

* *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 172.

of course depend on the dimensions of the structure. Reference to Mr. Duryea's discussion shows that his formula for the deflection of a cantilever element is identical with Mr. Noetzli's Equation (3), and that his formula for arch deflection may be translated into Mr. Noetzli's notation to read

$$D = \frac{2}{E} R f_c,$$

whereas Mr. Noetzli used the form,

$$D = \frac{1.56}{E} R f_c.$$

It may be noted here that if the author's formula for arch deflection should be found to be more accurate than that used in the Shoshone Dam analysis, it might be introduced into that analysis by making the Reclamation Service formula read:

$$K_a = \frac{100\,000\,x^2 - 308\,x^3}{1.56\,R^2\,(300 - x) - 4.92\,x^3 + 1\,599\,x^2},$$

the change affecting only the first term of the denominator.

If the Reclamation Service formula in its original form is applied to a section 100 ft. from the base of the Shoshone Dam, using $R = 140$, the resulting value for K_a is 36.70. If, instead, Mr. Noetzli's deflection coefficient is introduced, making the coefficient of the first term of the denominator 1.56 instead of 2, the resulting value of K_a is 40.25; that is, at Elevation 100 the arch carries $\frac{36.70}{62.50}$ or $\frac{40.25}{62.50}$ (57.7% or 64.4%) of the hydrostatic pressure, depending on the coefficient used. The value, 57.7%, agrees with the "water-pressure diagram" of the Reclamation Service chart. It is evident that if the author's coefficient was used, the division line between arch pressure area and cantilever pressure area, Fig. 14, would be shifted somewhat to the right, showing more load on the arches.

When the Reclamation Service formula for K_a is applied to a section 250 ft. above the base ($R = 148$), the corresponding results are:

$$K_a = 56.9 \text{ or } 58.4 \ (91\% \text{ or } 93.4\%)$$

depending on the coefficient of R^2 in the formula.

At the top of the dam, K_a becomes 61.8 or about 99% of hydrostatic pressure, but it is seen that there is no evidence in this analysis of arch action corresponding to a pressure in excess of hydrostatic pressure.

In computing the division of pressure at Elevation 250 of the dam by the use of the formula of the Shoshone Dam analysis, it may be urged that an error is introduced by considering the load carried by the cantilever element as that of Triangle *DCB*, Fig. 14, which is evidently too small, as shown by the form of the distribution curve below that elevation. This error is inherent in the formula, since it considers the pressure proportion, K_g, at any level to exist also below that level, giving a triangular distribution of load on the cantilever element. A better approximation for Elevation 250 apparently would be obtained by considering the load on the cantilever element as that shown by the triangle, *DBF*, with an area

four times as large as that of the triangle, *DBC*. If the computation is revised on this assumption, there is obtained:

$$K_a = \frac{100\,000\,x^2 - 308\,x^3}{0.5\,R^2\,(300 - x) - 4.92\,x^3 + 1\,599x^2} = 60.8,$$

or 97.3% as compared with 91% from the original formula.

In other words, the formula used by the Reclamation Service in developing the pressure distribution of its chart, may be regarded as a first approximation which might be corrected, as the computations proceed from bottom to top of the section, by altering the coefficient of R^2 in the denominator of the general formula for K_a at any level to conform more nearly to the distribution already computed for the lower levels; but however much the coefficient of R^2 is decreased to conform to the larger cantilever pressures below, the value of K_a cannot exceed 62.5, corresponding to hydrostatic pressure.

A. F. PARKER,* M. AM. SOC. C. E. (by letter).—It is evident that in this paper the author has presented to engineers some considerations of dam design which must be accepted as steps forward. The paper marks an evident "point of departure" for a much further advance to be made.

An engineer busily engaged in active work cannot take the time for original investigations; neither will clients willingly pay the expense of such work. He must often depend on results of research and investigations made by those who have the opportunity. The author has made such a clear exposition of principles and has simplified the formulas so that it seems to the writer a comparatively easy matter for one having a dam problem to solve, to start on the investigation of stresses and the design of arched dams without fear of an unreasonable amount of effort to reach understandable and satisfactory results.

The writer, as one who has squarely faced the practical problem of arched dams and has others now confronting him, is particularly interested in this paper. He pleads guilty to relying on the cylinder formula for the design of important arched dams, when it was realized that such formula was not fully satisfactory; but, nevertheless, the results were satisfying and successful. Before using the cylinder formula, all that could be found in the way of published investigations was studied and applied to the work in hand, but it must be admitted that the results were not convincing, probably because of the deeply involved mathematics. It was found, however, that when a dam section or thickness was determined by the cylinder formula with a unit pressure or water load of less than about 20 tons per sq. ft., all the more abstruse formulas gave results falling within the limits set by the simpler one.

The conditions of the problem of dam design vary with the peculiarities of each site, and, further, many of the fundamentals of the formulas are absolutely indeterminate, so it must be admitted that there is no possibility of exact solutions. The limits of safety in an arched dam of reduced section will remain largely a matter of speculation until some such structure fails or shows signs of failure. The need for some accurate observations of the action of curved dams, under varying conditions of water pressure and temperature, is great. There seems to

* Ogden, Utah.

be no method of testing the accuracy of mathematical determinations without the practical checks such observations will afford. No curved dam has failed as yet, or even showed signs of weakness.

The Crowley Creek Dam, in Malheur County, Oregon,* is reported by A. J. Wiley, M. Am. Soc. C. E., as being designed with a radius of 70 ft., the other dimensions and the stresses given being shown by his Table 2, which indicates a maximum stress of 440 lb. per sq. in. in the concrete 50 ft. below the crest, where the thickness is only 3.6 ft.

It is said that the attachment of the structure to the rock of the bottom and sides is not thoroughly done, that the quality of the concrete is far from good, and that (this is the interesting part), in the finished dam, "the alignment was so seriously at fault that the radius at the top varied from a minimum of 58 ft. to a maximum of 80 ft." Further, there was a contraction crack ⅜ in. wide at the top when the dam was empty in cool weather, but this crack closed as the reservoir filled; nevertheless, floods to a depth of 2 ft. had passed over the top of the dam, carrying "large blocks of thick ice," and still the dam did not fail.

It would be very interesting to have the stability of this arch accounted for mathematically. The fact that such a thin, indifferently built structure, so badly out of alignment, has successfully stood such a test, would seem to indicate that there is a larger "factor of safety" in thin arches than one could imagine, let alone show by calculations. This example cannot fail to cause wonderment as to what load of water pressure a well built, true, and symmetrical thin arch really could safely carry.

It is clearly proven that the less thickness in the arch the more definite, accurate, and reliable are the results of calculations of shears, shrinkage, temperature, and tensile stresses. It is probable that a more scientific and economical design than that afforded by the cylinder formula can and will be made by following up the way pointed out by Mr. Noetzli. The "cylinder hoof" theory certainly sheds much needed light on the matter.

As one who has had considerable experience with the practical design of arched dams the writer wishes to thank the author of this valuable paper for pointing out a method whereby it is evident that safer, more scientific, rational, and economical designs of arched dams may be made.

L. R. JORGENSEN,† M. AM. SOC. C. E. (by letter).—Undoubtedly, it is true that the theory of arch dams can be improved upon, but before any radical improvements can be introduced the elastic theory itself as applied to arch dams must be considerably modified.

As now applied, the elastic theory and actual practice do not agree. Theory may state that an arch dam should lift away from the bottom, or crack horizontally along the up-stream face with reservoir full, but the facts are that no horizontal cracks of any consequence are ever present. If horizontal cracks were present near the bottom of a dam with reservoir full, such reservoir would eventually empty itself through the customary vertical drain pipes between the center of the dam and the up-stream face.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 609.

† San Francisco, Cal.

It is of interest, therefore, to try to investigate the reason why the elastic theory cannot be applied to arch dams without modification.

When the load condition of an arch dam is considered, that is, a steady load coming on the structure gradually, sufficient data are available to show that the modulus of elasticity of the concrete is not constant, but changes with time and amount of stress. If a time interval of 14 days or more is interposed between the occurrence of no stress and a high stress, the modulus of elasticity will have changed in such a way that the concrete will either have stretched about three times or have compressed about three times as much as it would have if the same stress had come on fairly suddenly, as on a bridge.

This phenomenon of additional stretching or shortening after a time interval takes place where the dam material (concrete) is subject to high stresses, as it may be at various places in an arch dam or in many other structures. This time effect on the elasticity of the concrete goes far to relieve such possible high stresses and to prevent such ruptures as the author's calculations show should have taken place, for instance, on the Lake Spaulding Dam.

The writer has also noticed the action of the "time factor" on thin multiple-arch dams, where, according to the elastic theory, calculations would show that the arches should have cracked in the tension side, due to insufficiency of steel for temperature compensation and rib shortening. No cracks ever occurred, the time interval between the occurrence of low stress and high stress was simply long enough for the concrete to stretch additionally without cracking.

In a reinforced concrete floor-slab, the deformation will keep increasing for the first few weeks, if highly stressed, before it finally ceases. The steel in the tension side will not stretch more with time, but the concrete in the compression side will compress more, allowing additional sagging of the slab with time.

So far most investigators of the performance of arch dams (the writer included), have attached great importance to finding the portion of the load carried by the arch, and the portion carried by the cantilever. Recently, the writer has come to the conclusion that this is perhaps not so important, and is practically impossible of solution; it is simply too complicated to admit of close calculation.

This division of load between arch and cantilever for the middle portion of the dam is one thing, and the only thing, which the author has investigated, but it is entirely different along the arc as the abutments are approached. For the last 20% of the distance between the crown and the abutments, the cantilever action is even reversed when the dam acts principally as an arch.

In the writer's opinion, the simple arch formula is not entirely inexact, and as an arch dam can be made so that the structure will act as an arch from the start, and only be subject to a small deformation, this formula will give quite accurate results, except close to the bottom, where the structure should be investigated for shear and beam action.

The author objects to grouting the contraction joints, fearing that such practice will offset all the advantages of good and safe design, and continues:

"On the other hand, if a dam is closed at a low temperature * * *, a rise in temperature will produce bending moments, which eventually will cause other and perhaps still more dangerous cracks."

The writer cannot agree with this opinion. The "good and safe design" requires that arch action take place, but such arch action cannot and does not take place according to designing formulas unless the contraction joints are closed during times of low temperature. It is an ideal condition whenever the crown can be pushed up stream with reservoir empty in warm weather; then the movement of the crown on either side of the vertical will be a minimum, and it will be easy to make the structure water-tight and to make it stay water-tight. A water-tight dam can be expected to last longer than a leaky one.

The author claims that, on account of vertical arch action, gravity dams across narrow canyons cannot be safely considered to depend for their stability on gravity action alone. It is hard to understand how there can be any vertical arch action at all in a straight gravity dam, unless the dam has been "closed at a low temperature", and if it has, then the horizontal beam action certainly will give it considerably more strength than it possessed before the contraction joints were grouted.

The large deformation of the Lake Spaulding Dam referred to by the author occurred the first time the reservoir was full. It was thought when the so-called zero point was set that all the chemical heat was out of the structure, since the contraction joints had started to open, but later events indicated that the openings were only on the outside; contact still occurred farther inside the dam body, and the crown was actually pushed up stream when the zero position was set. After the reservoir had been full once and the dam had taken its permanent set, lost its chemical heat, and the shrinkage was completed, the contraction joints opened to more than $\frac{1}{4}$ in. The second time the reservoir was filled the crown of the dam near the crest deflected less than 1 in.

The contraction joints, with the exception of two, have since been closed by grouting and the deformation near the crest of the present dam is now $\frac{3}{16}$ in. maximum, with reservoir full, in the summer.

The contraction joints on the Spaulding Dam were not especially prepared for grouting purposes at the time the dam was built, and in order to grout these joints, holes had to be drilled from the outside faces, and the grout forced in. On account of the short time interval in which this grouting can be done, only one joint can be grouted per year.

The total leakage to-day through the dam amounts to about 1 gal. per sec., mostly through the ungrouted joints, with a little coming up through the vertical drain pipes (weepers) from the bottom of the foundation. Another gallon per second leaks out of the rock on the south side of the canyon, where the pressure outlet tunnel runs parallel to the side-hill; this leakage of course has nothing to do with the dam.

The author claims that according to his calculations horizontal cracks of considerable size must have opened on the water side of the dam, aggregating probably not less than $\frac{1}{2}$ in. at the up-stream face. If such cracks had opened, then each of the 6-in. drain pipes, located in the up-stream third of the section, would have been discharging several cubic feet of water per second with reservoir full, and the dam would be useless, as such cracks could not be closed.

At the stage of construction of which the author speaks, that is, when the Spaulding Dam was 225 ft. high, the arch section was practically equal to a gravity section, and, therefore, no horizontal cracking could very well take place, nor did it take place. On account of the presence of the contraction joints, the arch could not pull the cantilever back beyond its normal cantilever deflection, but this cantilever could have been pushed up stream by the chemical heat in the first place, and that is evidently what happened, since the large total deflection of the crown at the crest only occurred once, when the dam was first built.

Before long there may be available additional test data to show more fully the influence of the time factor on the modulus of elasticity of concrete under higher stress, and then, perhaps, a fairly accurate theory concerning arch dams can be developed. Such a theory must include the action of Poisson's ratio, shear, and beam action, which are as important as the cantilever action if it is to be claimed that such theory is more accurate than the simple arch formula.

It is the writer's strong conviction that the arch must be "closed at low temperature intentionally", in order that a true arch dam can be created.

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).—The author deserves much praise for his timely and illuminating paper. It should lead to clearer views on, and better designs of, arched dams. The theory is based on the principle that the deflections of the supposed horizontal arches and the vertical cantilever at the crown of the arches must be the same at equal depths. This co-ordination is effected by a tentative method.

In what follows, the dam is supposed to be divided into a series of horizontal arches, each 1 ft. in depth, and the hypothesis is made that any shear, either radial or axial, can be neglected, so that each arch can be regarded as free to move independently of the other arches. The vertical cantilever of the dam is supposed to be enclosed between two vertical radial planes, which likewise enclose the crowns of the arches, the angle between the planes being infinitesimal. The planes can then be regarded as parallel and the water pressure as acting parallel to them on the up-stream face of the cantilever, taken as plane, and perpendicular to the vertical planes mentioned.

If the thickness of such a cantilever is magnified so as to become 1 ft., the unit water pressures of the first become total water pressures of the second, and the conditions of equilibrium are unchanged. Such a cantilever, 1 ft. thick, on which the water pressures are all parallel, will be invariably considered in what follows. Its deflection curve, for given forces, can be readily found by the usual construction. For the arches, the author, by an ingenious method, has deduced very simple formulas for the deflection at the crown, whether from a given stress or water pressure, or for temperature changes and shrinkage.

With reference to the author's fundamental Equation (12), it seems advisable to tabulate values of the factor, $\frac{3}{16}\frac{L^2}{h}$, for various central angles, varying, say, from 40 to 120°, as it would lead to greater accuracy to use the correct values in place of a rough average.

* Chapel Hill, N. C.

Using the value 8.3 R for $\frac{L^2}{h}$, the author deduces the formula, Equation (13), for deflection at the crown of an arch,

$$D = \frac{1.56}{E} R f_c$$

where, from Equation (12), it is seen that f_c is the average stress on the cross-section of the arch considered.

The usual cylinder formula, $f_c t = P R_u$, is then introduced to derive the important Equation (13*a*). This formula for f_c is admittedly inexact, although its use is so necessary to effect a solution in the analytical investigation of a curved dam. There is no such necessity in the method proposed by the author, and it seems to the writer that the time has come to write for f_c a more correct value. This is found by regarding the horizontal arch as a "thick cylinder" and applying the corresponding formula for circumferential unit stress*.

The writer has given in his discussion of the paper by B. A. Smith, M. Am. Soc. C. E., on "Arched Dams"† a close approximation to the arch thrust ($f_c t$), when t is so small that it can be neglected when added to R_1, the radius of the inner surface of the arch‡. In the author's notation, this total arch thrust is,

$$f_c t = \frac{P R R_u}{R_1}; \text{ or } f_c = \frac{P R_u}{t} \frac{R}{R_1},$$

where, for any horizontal arch,

P = unit water pressure at mid-depth of the arch;
R_u = up-stream radius of arch;
R = radius of the center line, so that $R = \frac{1}{2}(R_u + R_1)$;
R_1 = inner, or down-stream, radius of arch;
t = thickness of dam or arch at its mid-depth.

Substitution of this value for f_c in Equation (13) gives

$$D = \frac{1.56}{E} \times \frac{P R R_u}{t} \times \frac{R}{R_1} = \frac{1.56}{E} \times \frac{P R^2}{t} \times \frac{R_u}{R_1}.$$

The author's value for D, the deflection at the crown, is seen to be increased in the ratio $\frac{R}{R_1}$. This is generally small. Thus, for the Wooling Dam, where $R_u = 135$ ft.; t, at the crest, $= 2.2$ ft., and at the base, $t = 4.4$ ft., the increase in D over the author's value varies from 0.8% at the crest to 1.7% at the base. For thicker dams, the increase would be greater. It is found desirable in practice to compute D to three significant figures, hence, it seems advisable to use the more correct formula. As to the coefficient, $\frac{5}{176}$, of Equations (11) and (12), and, consequently, the 1.56 of Equations (13*a*) and (13*b*), it is possible that a slight reduction is desirable, since the arches are not exactly free at the ends, and the deflection is smaller for a hingeless arch than for one hinged at the ends.

Preliminary to finding Equation (11), the author has deduced in Equation

* Such a formula is given in books on Strength of Materials, *e. g.*, Morley, Article 122, Equation (12).

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2084.

‡ See p. 80 of this discussion. Using the notation of the discussion, $r = r_1 + t$. $\therefore r^2 + r_1^2 = 2r_1^2 + (2r_1 + t)\ t = 2(r_1^2 + r_1\ t)$, nearly; $r^2 - r_1^2 = (2r_1 + t)\ t = 2r_1\ t$, nearly. These admissible approximations lead at once to the values given for the unit thrusts at the outer and inner surfaces and to the formula for T, the total thrust.

(10*a*) a formula for the length of a circular arc, in terms of the span and rise, which merits special notice, and may prove useful to engineers and mathematicians.

As introductory to the subject of "combined cantilever and arch action," the numerical results worked out by Mr. B. A. Smith for the Wooling Dam* will be utilized and analyzed with the object of finding how much of the load is carried by arch action and how much by cantilever action. A trial-and-error method for dealing with any arched dam will be suggested by these results, and, finally, such methods will be applied to the Wooling Dam, in order to compare the final results with those of Mr. Smith.

The Wooling Dam is 33 ft. high, the crest thickness 2.2 ft., the thickness at the base 4.4 ft., and the radius of the up-stream face, R_u, equals 135 ft. The angle of the sector at crest level is 120 degrees. For reservoir full, the water surface is supposed to be level with the crest of the dam. The dam is reinforced vertically and the base—tied to the rock foundation—is supposed to be fixed, or encastrée. There is no horizontal reinforcement; hence, since plain concrete cannot be counted on to stand more tension than, say, 150 to 200 lb. per sq. in., if this amount of tension is exceeded in the horizontal arches when shrinkage and fall of temperature are considered, then vertical cracks will be formed. Often, vertical construction joints are prescribed; in which case, there can be no tension across such joints, and they will open for a sufficient fall of temperature.

Now, suppose the dam is divided into the series of horizontal arches, each 1 ft. in depth, the mid-plane of each being at a depth, d ft., below the crest, and regard these arches as perfectly free to move over each other, all radial or axial shear between them being neglected. As previously explained, a vertical cantilever, 1 ft. thick, of the full height of the dam, will be considered at the crown of the arches. The dam will first be investigated for reservoir full and no shrinkage or change of temperature.

In Table 5, the arch thrusts, T, at the successive depths, d, are taken from Mr. Smith's Table 4, and the unit water pressures, p, acting on the successive arches, as computed from the cylinder formula, $T = p\,R_u$, or $p = \dfrac{T}{135}$, are inserted in the third column of Table 5. In the fourth column are given the full unit water pressures, $p' = 62.5\,d$, at the successive depths, d, and the last column gives the unit pressures, $p' - p$, on the cantilever.†

The cylinder formula, $T = p\,R_u$, was used because of its simplicity. However, as Mr. Smith's analysis includes radial shear between the successive horizontal arches, the numerical values of p and $p' - p$, given in Table 5, are only closely approximate. It will be observed, from a consideration of the figures in Table 5, that from $d = 7.3$ to the base, at $d = 33.0$, the horizontal load on the cantilever, $p' - p$, is positive and acts down stream, so that both the arches and the cantilever are acted on by forces directed down stream at such levels. At $d = 0$ or $d = 3.6$, $p' - p$ is negative; hence, the load on the cantilever acts up stream. This shows that at the crest, $d = 0$, the cantilever, acting alone under the water pressure, would deflect more than the horizontal arch at that point. This excess

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 2068, 2075, Tables 4 and 6.

† Table 5 is taken from the writer's discussion of Mr. Smith's paper on "Arched Dams," previously cited.

deflection is resisted by the arch, which reacts on the cantilever, and thus supplies the force acting up stream on the cantilever. Similarly, at $d = 3.6$, the arch reacts on the cantilever with a force, directed up stream, of 132 lb. per sq. ft. By interpolation, it is found that at $d = 5.4$ ft. there is no pressure on the cantilever, so that at that depth the entire water pressure, $62.5 \times 5.4 = 337.5$ lb. per sq. ft., is carried entirely by the horizontal arch. This point, marked K in the diagrams, plays an important rôle in the tentative solution proposed by the author.

TABLE 5.—RESERVOIR FULL, NO SHRINKAGE OR CHANGE OF TEMPERATURE.

d, in feet. (1)	T, in pounds per foot of height. (2)	$p = \frac{T}{185}$, in pounds per square foot. (3)	$p' = 62.5\,d$, water pressure, in pounds per square foot. (4)	$p' - p$, pressure on cantilever, in pounds per square foot. (5)
0	49 000	363	0	− 363
3.6	48 200	357	225	− 132
7.3	45 800	340	456	+ 116
11.0	41 100	305	687	+ 382
14.6	35 500	263	912	+ 649
18.3	27 600	205	1 144	+ 939
22.0	18 700	139	1 375	+ 1 236
25.6	9 900	73	1 600	+ 1 527
29.3	2 900	21	1 831	+ 1 810
33.0	0	0	2 062	+ 2 062

NOTE.—When the pressure, $p' - p$, on the cantilever is negative, it acts up stream; when positive, it acts down stream.

Fig. 15 shows a vertical section of the cantilever, $A\,F\,G\,B$, which is 1 ft. thick perpendicular to the plane of the paper. The water is to the right of the vertical face, $F.\ G.$ The pressures, $p' - p$, on the cantilever are laid off as horizontal ordinates from $F\,G$, to the left when positive, to the right when negative. A dotted line is drawn through the extremities of these ordinates. If a straight

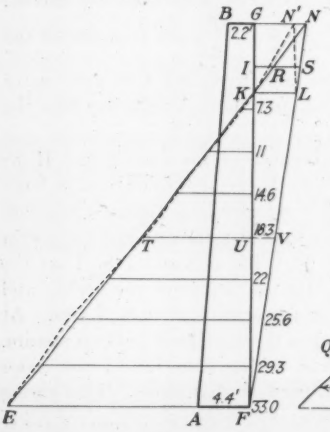


FIG. 15.

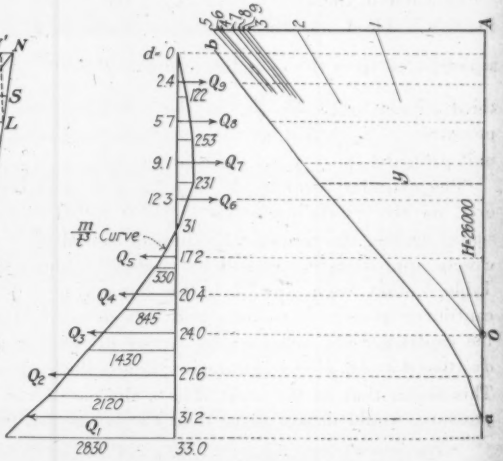


FIG. 16.

line is drawn from E to where the dotted line crosses $F\,G$ at K and produced to N, on a level with the crest, it is seen nearly to coincide with the dotted line except near the crest, and may be taken for J as a first approximation. $F\,E$ is always laid off, to scale, equal to the full water pressure, in pounds per square foot, at the base, since arch action cannot occur there, and the cantilever has to carry the entire load. The pressure on the cantilever at the crest is really $G\,N'$, so that if $G\,N$ is substituted for it, it is foreseen that the computation to be given presently, will give an excess pressure at the crest.

As a first approximation, assume the straight line, $E\,K\,N$, as limiting the pressures, or as replacing the dotted line. It will be observed that if the straight line, $F\,N$, be drawn, the horizontal ordinates of the triangle, $E\,F\,N$, will give, to scale, the water pressures, p', at the successive depths, d, of Table 5. It follows that the horizontal ordinates of the triangle, $F\,G\,N$, represent the pressures, p, on the arches, acting to the left. Thus, at U,

$$p' - p = V\,T - V\,U = U\,T,$$

acting to the left, and at I,

$$p' - p = S\,R - S\,I = I\,R,$$

acting to the right; giving, respectively, the pressures on the cantilever at U and I. Further, the total pressure on the cantilever is represented by $E\,F\,K - K\,G\,N$, and that on the arches by $F\,G\,N$ or by $F\,K\,N + K\,G\,N$. On adding, the total water pressure, $E\,F\,N$, is obtained as it should be.

Now, suppose that no computations had been made for this dam, and it was desired to find approximations to the thrusts, moments, and deflections. The procedure would be as follows:

Lay off to scale, as in Fig. 15, $F\,E = 2\,062$ lb., the total unit water pressure at the base, assume the position of point K, and regard the straight line, $E\,K\,N$, as approximately limiting the pressures on the cantilever; then, for such pressures, by the usual graphical construction, find the deflection of point K of the cantilever, and compare it with the deflection of the horizontal arch at that point, as computed from the author's Equation (13a), corresponding to the full water pressure, $L\,K$, at the point. If the agreement is not sufficiently close, assume another point, K, and repeat the procedure until the deflections for arch and cantilever are the same. Closer results will be obtained by guessing at a point, N', and drawing the straight line, $K\,N'$, to limit the pressures above K. The line, $L\,N$, is now to be replaced by $L\,N'$. The labor of computation is not increased thereby, and it is better to follow this procedure from the start.

The foregoing is seen to be practically the author's method, which was deduced without any aid from Mr. Smith's illuminating results and, thus, is entitled to all the more credit.

For dams of other proportions than the Wooling Dam, it is possible that the straight line, $E\,K$, will not sufficiently agree with the pressure curve. If so, a curved line to satisfy the conditions must be found by trial. Such conditions are that the deflection curve for the cantilever, as found from Fig. 15, must agree closely with that for the arches, as computed from Equation (13a) for a number of points.

As preliminary to the graphical construction for finding the horizontal deflections of the cantilever, it is necessary to find the sum of the moments of the successive pressures on the cantilever above a point on $F\,G$, as I or U, about that point. Thus, if the line, $E\,K\,N$, Fig. 15, is supposed to limit the pressures, the moment m about I is equal to the area of the trapezoid, $I\,R\,N\,G$, multiplied by the distance from its center of gravity to $I\,R$. Similarly, the moment about K equals $\frac{1}{2}\left(K\,G \times G\,N\right) \times \frac{2}{3}\,K\,G$. If the line limiting the pressures is taken as $K\,N'$ in place of $K\,N$, simply replace N by N' in the computations.

Consider, next, the triangle, $K\,U\,T$; then, if $K\,U = x$, $K\,F = e$, $E\,F = c$, we have $U\,T = \frac{c}{e}x$, and the moment of the pressure triangle, $K\,U\,T$, about U is,

$$\frac{1}{2}\,x\,.\,U\,T\,.\,\frac{1}{3}\,x = \frac{1}{6}\,\frac{c}{e}\,x^3.$$

The moment, m, of all pressures on the cantilever above U about U is, therefore,

$$m = \frac{1}{6}\,\frac{c}{e}\,x^3 - \frac{1}{2}\left(K\,G \times G\,N\right)\left(\frac{2}{3}\,K\,G + x\right),$$

on remembering that the pressures from K to U act to the left, whereas those from K to G act to the right.

For a first trial, assume the point, K, to be 5 ft. below the crest; hence, when $d = 5$, $x = 0$. The moments, m, for the depths, d, below the crest were computed from the foregoing formulas and are inserted in Table 6. The values of the thickness, t, of the dam at the successive depths, d, are inserted in the fourth column, the values of t^3 in the fifth column, and the values of $\frac{m}{t^3}$ in the sixth column.

TABLE 6.—WOOLING DAM, WATER PRESSURE ONLY, FIRST TRIAL: QUANTITIES FOR CANTILEVER.

d, in feet. (1)	x, in feet. (2)	m = moment at depth d, in foot-pounds. (3)	t, in feet. (4)	t^3. (5)	$\frac{m}{t^3}$ (6)	d, in feet. (7)	Load, $Q = \frac{M\,.\,\Delta X}{t^3}$ (8)
0	....	0	2.20	10.6	0	2.4	$Q_9 = -$ 220
3.6	....	− 1 810	2.46	14.9	− 122	5.7	$Q_8 = -$ 690
7.3	2.3	− 5 030	2.71	19.9	− 253	9.1	$Q_7 = -$ 940
11.0	6.0	− 5 930	2.95	25.7	− 231	12.3	$Q_6 = -$ 470
14.6	9.6	− 1 010	3.18	32.2	− 31	17.2	$Q_5 = +$ 540
18.3	13.3	+ 13 210	3.42	40.0	+ 330	20.4	$Q_4 = +$ 2 170
22.0	17.0	+ 41 730	3.67	49.4	+ 845	24.0	$Q_3 = +$ 4 180
25.6	20.6	+ 85 510	3.91	59.8	+ 1 430	27.6	$Q_2 = +$ 6 550
29.3	24.3	+ 151 070	4.15	71.5	+ 2 120	31.2	$Q_1 = +$ 9 140
33.0	28.0	+ 241 180	4.40	85.2	+ 2 830	....	....

The values of $\frac{m}{t^3}$ are now plotted as horizontal ordinates at the corresponding points of the vertical side of the cantilever, Fig. 16; and the areas, Q_1, Q_2, * * *, between ordinates are found, as well as the depths, d, of the centers of gravity of these areas, through which Q_1, Q_2, * * *, are supposed to act horizontally, to the left or right according as m is positive or negative. Then, with a convenient pole, O, lay off vertically, to any scale, the pole distance,

$H = OA = 26\,000$; and from A, to the same scale, lay off horizontally to the left, the loads, Q_1, Q_2, * * *, Q_9 (the subscripts only are given in Fig. 16) in order, to the left or right, according as Q acts to the left or right. The equilibrium polygon, $a\,b$, is then drawn in the usual manner, and it will be shown that the horizontal ordinate, y, measured to the scale of distance, in feet, from $a\,A$ to the equilibrium polygon, $a\,b$, divided by 100, is the deflection, in inches, of the cantilever at the corresponding depth. The equilibrium polygon, $a\,b$, is thus the magnified deflection curve of the cantilever.

This corresponds to Mohr's graphical method, but reference will be made to Morley's "Strength of Materials", Art. 83, for a convenient formula for deflection. In this, $I = \frac{1}{12}\,t^3$ is the moment of inertia of the cantilever where the thickness is t, $\frac{M}{t^3}$ is the average ordinate for the vertical depth, $\Delta\,x$, between the ordinates of the $\frac{m}{t^3}$ curve of Fig. 16 considered, so that generally, $Q = \frac{M\,\Delta\,x}{t^3}$. Further, x represents the vertical distance from a point the deflection of which is desired, to any load, Q, below that point, and E = modulus of elasticity of concrete = $2\,600\,000 \times 144$ lb. per sq. ft., as given by Mr. Smith. Thus, the deflection of the point (from which x is reckoned) is, in feet,

$$\frac{1}{E}\,\Sigma\,\frac{M\,\Delta\,x}{I}\,x = \frac{12}{2\,600\,000 \times 144}\,\Sigma\,\frac{M\,\Delta\,x}{t^3}\,x,$$

and, in inches,

$$\Delta_x = \frac{1}{2\,600\,000}\,\Sigma\,Q\,x,$$

in which the sum of the products of the type, $Q\,x$, must include all loads between the point and the base of the cantilever. This sum is readily computed for any one point, but to find the deflections of all the points, or the deflection curve, the graphical method is far superior. By this method, if y is the horizontal ordinate to the equilibrium polygon of Fig. 16, at the point from which x is measured, then, by a well known principle of this polygon,

$$\Sigma\,Q\,x = H\,y = 26\,000\,y.$$

Hence, substituting in the foregoing equation

$$\Delta_x = \frac{y}{100}\text{ in.}$$

where, as previously mentioned, y must be measured, to the scale of distance for the cantilever, in feet.

For this first trial, only the deflection at the point, K, 5 ft. below the crest, is desired. From a large scale drawing, this is found to be 0.191 in. The deflection for the horizontal arch at point K, where the full water pressure, $P = K\,L = 312$ lb. per sq. ft., acts on the arch, is found by substituting in the author's Equation (13a), $t = 2.56$, $R_u = 135$, $R = 133.7$, $E = 2\,000\,000 \times 144$*, to be 0.143 in. Since the deflection at K is greater for the cantilever than for the arch, the point, K, must be lowered.

* In the Wooling Dam, there is vertical reinforcement but no horizontal reinforcement. On that account, Mr. Smith takes $E = 2\,000\,000$ lb. per. sq. in. horizontally, or for the arches, and $E = 2\,600\,000$ lb. per sq. in for the cantilever.

For a second trial, let K be placed 6 ft. below the crest. On repeating the entire computation for this position of K, it is found that the deflection there of the cantilever = 0.135 in., and of the arch 0.169 in. The simple construction of Fig. 17 would seem to indicate that K should next be placed at d = 5.6 or 5.6 ft. below the crest. The result evidently would have been closer if the point, N, of Fig. 15, had been shifted to N' in both trials; for it will be found that slight changes produce a marked effect on some of the quantities, particularly deflections.

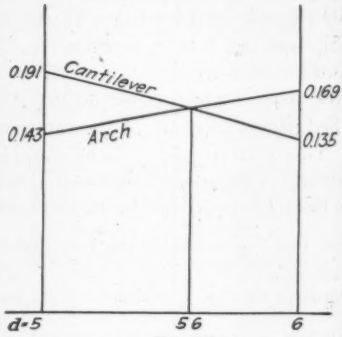


FIG. 17.

For a third trial, K was placed 5.6 ft. below G and GN' was laid off, to scale, as 364 lb. per sq. ft., Fig. 15. The straight lines, KN' and LN' being drawn, it will be seen that the total pressure on the cantilever is $FEK - KGN'$, and on the arches is $FKL + LKGN'$, the sum of which, $FLKE + LKN'$, represents the total water pressure on the vertical face of the dam, FG. The horizontal ordinates of these areas give the corresponding forces acting; those on the cantilever being directed to the left in area FEK and to the right in area KGN', whereas all the forces acting on the arches in the area, $FKGN'LF$, act to the left.

The computations and cantilever deflection diagram are effected as before, so that only the final results are given in Table 7, in which the more exact values obtained by Mr. Smith are added by way of comparison.

TABLE 7.—WOOLING DAM, WATER PRESSURE ONLY: THIRD TRIAL.

d, in feet. (1)	Moments on cantilever, in foot-pounds. (2)	Moments, M (Smith), in foot-pounds. (3)	Deflection of cantilever, in inches. (4)	Deflection of arches, in inches. (5)	Deflection (Smith), in inches. (6)	Arch thrust, $T = 135\ P$, in pounds. (7)	T (Smith), in pounds. (8)
0.0	0	0	0.198	0.195	0.188	49 100	49 000
3.6	— 1 850	— 1 900	0.175	0.169	0.166	47 900	48 200
5.6	.....	.....	0.162	0.159	0.154	.....	.....
7.3	— 5 530	— 5 500	0.150	0.142	0.143	44 300	45 800
11.0	— 7 430	— 7 500	0.125	0.112	0.118	37 900	41 100
14.6	— 3 870	— 4 000	0.098	0.087	0.093	31 700	35 500
18.3	+ 8 870	+ 7 400	0.071	0.065	0.068	25 500	27 600
22.0	+ 34 600	+ 32 600	0.045	0.045	0.042	19 000	18 700
25.6	+ 75 800	+ 73 200	0.022	0.029	0.021	12 900	9 900
29.3	+ 138 300	+ 134 800	0.006	0.014	0.006	6 600	2 900
33.0	+ 225 400	+ 220 900	0.000	0.000	0.000	0	0

The deflection for the arches, given in the fifth column, was computed from the author's Equation (13a), which reduces, for R_u = 135, $E = 144 \times 2\,000\,000$ lb. per sq. ft. to

$$D = 0.00000878 \frac{P\,R}{t},$$

in inches. At the depth, d, t is the thickness of the dam and R the radius of the center line, both in feet, and P is the pressure on the arch, in pounds per

square foot. The values of P may be computed or measured, to scale, from the horizontal ordinates of $F\,G\,N'\,L\,F$, Fig. 15. Finally, the arch thrust for the width, t, or $T = 135\ P$, is given in Column 7 of Table 7. From the near equality of the values of the deflections of the arch and the cantilever at $d = 0$ and $d = 5.6$, it is seen that the points, N' and K, are (practically) correctly located, so that the values of P are nearly exact down to the point K. Below K they are approximate, since the line, $E\ T\ K$, is not straight, as assumed. In fact, if, in Fig. 15, $U\ T$ is the ordinate to the straight line, $E\ K$, but $U\ T'$ is the ordinate to the dotted (true) curve, then $U\ T'$ is the unit pressure on the cantilever, and subtracting it from $V\ T$, the water pressure, the true pressure on the horizontal arch at U is equal to $V\,T - U\,T'$. This is equal to $U\,V$ only when T and T' coincide. The pressure on the arches below K must be found in this way, and not from the triangle, $F\ K\ L$, whenever it is found desirable to consider some dotted curve, $E\ K$, in place of the straight line, in making further trials.

Since the deflection of an arch varies directly with P, and since the deflections of the arches from $d = 7.3$ to $d = 22.0$ are less than, and from $d = 22$ to the base, greater than, the deflections of the cantilever, it is seen that P must be increased over the first interval and decreased over the second; or, since $P = V\,T - U\,T'$, the dotted curve must fall below the straight line, $E\,K$, Fig. 15, from $d = 7$ to $d = 22$ (about) and above it from $d = 22$ to the base. This is in exact agreement with Mr. Smith's values of P, as seen by plotting to a large scale.

The question now arises whether the values in Table 7 are not sufficiently exact, as a practical solution, for this dam. Whatever answer may be given, the designer will hardly rest content without further trials to equalize more exactly the deflections for the arches and cantilever, particularly as a new trial, with checking, need not take more than eight hours. To reach approximately correct results most quickly, it is plainly desirable to assume some curve, as the dotted curve, from E to K and also the point, N', in the first instance, using the results for the Wooling Dam as some guide in locating the curve and the point, N'.

Temperature Changes.—The influence of temperature changes, from an assigned normal temperature, is clearly brought out in Mr. Smith's results pertaining to the Wooling Dam.* The entire dam was supposed to experience a rise in temperature of 20° Fahr. The method of treatment is in two steps, as follows:

1.—The base is assumed to be perfectly smooth and the dam is assumed to expand equally in every radial direction (or to have a "cylinder expansion"), so that the center line, $A\,D\,B$, Fig. 18 (*a*), takes the position, $A'\,C\,B'$, under the temperature rise of θ degrees Fahrenheit. Hence, if c is the expansion of concrete per unit of length for 1° Fahr. the arc, $A\,D\,B$, has expanded, in feet,

$$A\,D\,B\,.\,c\,\theta = R\,\phi\,.\,c\,\theta,$$

where R is the radius of the center line, $A\,D\,B$, and ϕ is the central angle, $A\,O\,B$, expressed in radians.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2075, Table 6.

The increase of length of the center line is, likewise, in feet,

$$A'CB' - ADB = (R + CD)\,\phi - R\phi = CD \cdot \phi.$$

Equating these two values for the increase of the arc, ADB, there is obtained

$$CD = cR \cdot \theta.$$

As Mr. Smith substitutes R_u for R throughout in his analysis (in the paper cited), the same will be done in what follows, so as to compare results on the same basis. This cylinder expansion is used because, after this first expansion, the dam, save for its weight, is without stress.*

2.—In the second step, the dam is supposed to be forced back under the action of radial forces, acting inwardly, such that the base is brought back to its original position, ADB, Fig. 18 (b), the cylindrical generators being vertical throughout the base, ADB. This condition must be fulfilled because the dam is supposed to be fixed at the base, with the tangent to the deflection line there vertical. Above the base, except at A and B, Fig. 18 (b), the arc, ACB, is not forced back so far and, at a certain height, the dotted arc, AIB, can represent the projection of the final position of the center line. Evidently, the forces that cause this inward motion of ACB are the reactions of the dam acting as

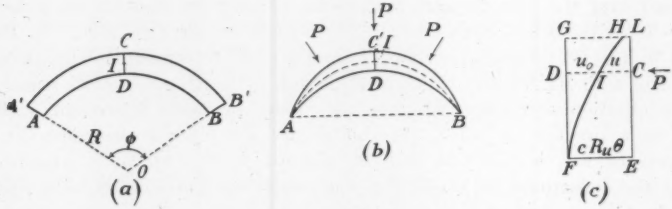


FIG. 18.

a cantilever on the dam acting as an arch. The arch expands under the temperature change and tends to move up stream. This is resisted by the dam acting as a cantilever, so that the normal forces, P, acting on any horizontal arch, are exactly the reactions (pulls, acting down stream for a rise of temperature) of the cantilever on the arch.

The movements indicated are clearly shown in Fig. 18 (c), which represents a vertical radial section at the crown, and at the crown only. In this diagram, FG represents the original position of a vertical line through D of Figs. 18 (a) and 18 (b); then, after the cylinder expansion, FG moves up stream to EL a distance, $cR_u\theta$. The horizontal arches are then subjected to the inward acting radial forces, having the same intensity, P, at the same level, which causes the point, E, to move back to F, the point, C, on the vertical, EL, to move to I, and the point, L, at the crest, to move to H, all motion corresponding to the theory hitherto developed for the arched dam. The elastic curve is represented by the line, FIH, having a vertical tangent at F. Finally, if for any point, C, on EL, $u = IC$ = deflection under the forces, P, from the expanded position of

* This would not be true if the ends, A and B, had been regarded as hinged, and the center line, ADB, had been shifted, under the expansion, to ACB, Fig. 18 (b). The dam, then, would have been under stress from arch action, and the next step would have been inapplicable.

the horizontal arch at that point, and $u_0 = D\,I$, then the actual deflection of the point, D, on $F\,G$ is $D\,I = D\,C - I\,C$ or,

$$u_0 = c\,R_u\,\theta - u.$$

At the crest, the deflection up stream is $G\,H$. It is not equal to $G\,L$, although the difference is relatively small.

The value of u, the deflection of a segmental horizontal arch, free to move, can be deduced from Mr. Smith's Equation (5),

$$T = \frac{2}{3}\,\frac{E\,t}{R_u}\,u;$$

or, since $T = P\,R_u$,

$$u = \frac{3}{2}\,\frac{R_u}{E\,t}\,T = \frac{3}{2}\,\frac{R_u^{\,2}}{E\,t}\,P.$$

Mr. Noetzli's Equation (13a), for the deflection, D, of the arch is:

$$D = 1.56\,\frac{R\,R_u}{E\,t}\,P,$$

which differs but slightly from the preceding formula, and is more accurate in so far that R, the radius of the center line, is used in place of R_u. As to the factor, 1.56, as already observed, it is more accurate to replace it with a term that varies with $\frac{L^2}{h}$ and tabulate its values for various central angles.

For a fall of temperature or shrinkage, the demonstration is similar. In the first stage, the radii are all supposed to decrease by an equal amount, $C\,D$, Fig. 18 (a). Then radial forces, acting outwardly, are applied, which force the base back to its original position, but all parts of the dam above the base will suffer a deflection down stream. The radial forces are simply the resistances of the cantilevers to the movement of the arches, and thus act up stream. If Fig. 18 (c) is revolved 180° about $F\,G$, then $F\,I\,H$ will represent the deflection line, the deflection at any point, D, being down stream and equal to

$$u_0 = C\,D - u,$$

where $C\,D$, the cylinder deflection, can be computed. It is $cR_u\theta$ for θ degrees fall of temperature, and $c'R_u$ for shrinkage, if c' is the shrinkage per unit of length of arc. Since the arches are now in tension, the theory is only valid when the arches and abutments can support the tension, which will generally require horizontal reinforcement of the dam.

Now, consider the Wooling Dam with reservoir empty and a rise of temperature of 20° Fahr. To ascertain the degree of accuracy that may be expected from the hypothesis of horizontal arches free to move, acting together with the vertical cantilever at the crown, the values of the arch thrusts, T (on the area, $t \times 1$) will be taken from Mr. Smith's Table 6, from which the values of the radial forces, $P = \frac{T}{135}$, can be at once tabulated, and the computations for moments and cantilever deflections effected as hitherto explained.

In Table 8, P is the unit pressure on the cantilever, acting up stream, or the reaction of a horizontal arch, at depth, d, below the crest, on the cantilever. The

cantilever, at the same depth, exerts an equal force, P, on the arch, acting down stream, as hitherto noted.

TABLE 8.—WOOLING DAM, TEMPERATURE RISE OF 20° FAHR., RESERVOIR EMPTY.

d, in feet.	T, arch thrust, in pounds.	$P = \frac{T}{135}$, in pounds per square foot.	d, in feet.	m, moment, in foot-pounds.	$\frac{m}{t^3}$	d, in feet.	Q.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0	3 150	23	2.14	78	6	1.40	Q_{10} = 6
3.6	10 700	79	5.63	640	38	4.31	Q_9 = 77
7.3	19 700	146	9.27	2 894	122	7.76	Q_8 = 291
11.0	29 900	222	12.90	7 471	260	11.30	Q_7 = 693
14.6	41 200	305	16.53	15 566	433	14.86	Q_6 = 1 258
18.3	53 100	394	20.21	28 520	638	18.49	Q_5 = 1 999
22.0	65 100	482	23.85	47 230	861	22.11	Q_4 = 2 730
25.6	76 300	565	27.48	72 750	1 110	25.74	Q_3 = 3 577
29.3	86 000	637	31.17	106 880	1 360	29.39	Q_2 = 4 557
33.0	92 900	688	33.00	128 290	1 510	32.10	Q_1 = 2 626

In Fig. 19, $F\,G$ represents the vertical face of the cantilever, and at the successive depths, 0, 3.6, * * *, the pressures, P, are laid off as horizontal ordinates, from which the total pressures for successive depths can be computed, and also the vertical ordinates of their centers of pressure. The sum of the moments of such pressures from the crest down to a point given in the fourth column by its value of d, is given in the fifth column, and the values of $\frac{m}{t^3}$

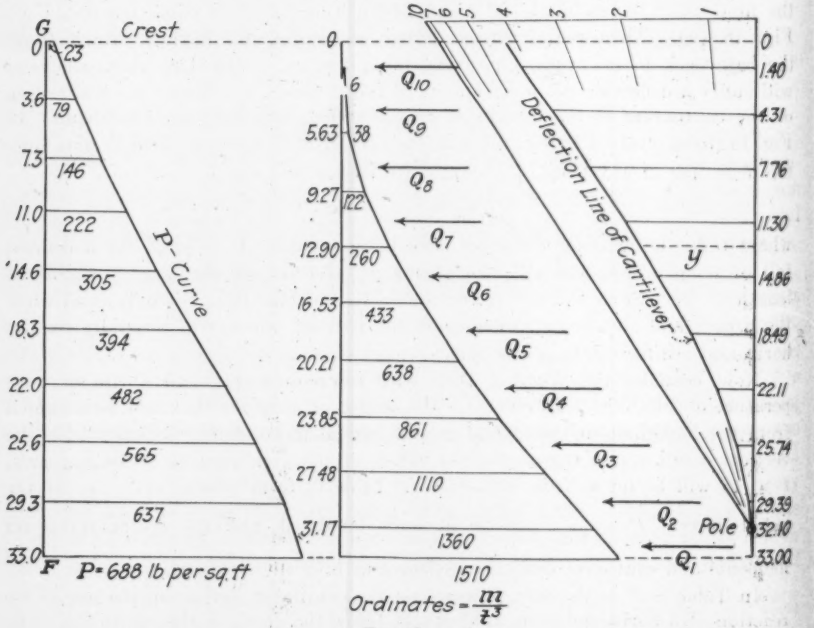


FIG. 19

in the sixth column. The latter values are then laid off as horizontal ordinates, Fig. 19, and the values of Q, as already defined, are computed and tabulated in the eighth column, and their lines of action computed and given in the seventh column. The deflection diagram is then drawn exactly as hitherto explained, using a pole distance of 26 000 to the same scale as Q. Any ordinate, y, to the scale of distance, divided by 100, gives the deflection of the cantilever, in inches, at the point where y is drawn. For accuracy, all lever arms were computed, and distances laid off, to scale, to hundredths of a foot. The comparison of results is given in Table 9.

TABLE 9.—COMPARISON OF DEFLECTIONS.

d, in feet.	Cantilever deflection, in inches.	u_0 = arch deflection, in inches.	Deflection as computed by Smith, in inches.
0.0	— 0.165	— 0.166	— 0.168
3.6	— 0.145	— 0.141	— 0.143
7.3	— 0.120	— 0.117	— 0.118
11.0	— 0.095	— 0.092	— 0.093
14.6	— 0.072	— 0.068	— 0.069
18.3	— 0.049	— 0.047	— 0.048
22.0	— 0.030	— 0.028	— 0.029
25.6	— 0.015	— 0.013	— 0.013
29.3	— 0.004	— 0.003	— 0.004
33.0	— 0.000	— 0.000	— 0.000

The minus sign before the deflections indicates that the motion, for the rise of temperature of 20° Fahr., is up stream. For a corresponding fall, the motion would be down stream and the deflections positive.

The values of u_0, the deflections for the horizontal arches, supposed to be free to slide over each other, are computed (in feet) from the formula already given:

$$u_0 = c\, R_u\, \theta - \frac{3}{2}\,\frac{R_u}{E\,t}\,T,$$

or, introducing the constants given by Mr. Smith, $c = 0.0000055$, $R_u = 135$, $\theta = 20$, $E = 144 \times 2\,000\,000$, and multiplying by 12 to reduce to inches

$$u_0 = 0.178 - 0.00000844\,\frac{T}{t}.$$

The practical agreement in the deflections as computed in three different ways is remarkable. It is equally so with regard to the moment curves which were plotted to a large scale. This proves that, for this arch, the hypothesis that the horizontal arches are free to slide over each other, is practically realized.

The unit pressure, P, at the base is the only value of P that can be computed independently. Since, for the base, Fig. 18 (c), $u_0 = 0$; on solving the formula for $T = P\,R_u$ and then dividing by R_u,

$$P = \frac{2}{3}\,\frac{E\,t}{R_u}\,(c\,\theta);$$

substituting numerical values, $P = 688$ lb. per sq. ft., as given in Table 8. If Mr. Noetzli's Equation (13a) is used, replace $c\,R_u\,\theta$ by the more accurate value, $c\,R\,\theta$, and thus obtain, for the base,

$$P = \frac{E\,t}{1.56\,R_u}\,(c\,\theta).$$

For a given curved dam, to find the thrusts and moments due to a change of temperature, the first step is to compute P at the base and lay it off at F, Fig. 19. Then, to use the tentative method, assume a very small value for P at the crest, G, and draw a P curve, similar to the one in Fig. 19, for a first trial. Then, with the pressures corresponding, proceed as in Fig. 19 to find the deflection line for the cantilever. Next, the deflections, $u = C\,I$, Fig. 18, for various values of d, must be found for the horizontal arches, subjected to the same unit pressures, P, as the cantilever at the same depths. These deflections can be computed by Mr. Noetzli's Equation 13 (a). Finally, as explained, these computed deflections are from the position, $E\,L$, Fig. 18 (c), of a vertical at the crown in the first expanded position; hence, the actual deflection of the crown from the original position of this vertical is $D\,I = D\,C - I\,C$, or

$$u_0 = c\,R\,\theta - u = c\,R\,\theta - \frac{1.56}{E}\,\frac{P\,R\,R_u}{t}.$$

If the P curve has been accurately drawn, the deflections of arch and cantilever should agree for the same depths, d; if not, a second trial must be made, and so on, until practical agreement is effected.

It is natural to inquire as to the percentage of error due to substituting a straight line for the P curve. This was done for the Wooling Dam, drawing the straight line through the extremities of the curve. The results show that, for the upper half of the dam, T was in excess by from 0 to 21% and M in excess by from 4 to 20%; whereas for the lower half of the dam, T was too small by from 0 to 5%, but M was in excess by from 5 to 11 per cent.

In the case of fall of temperature and shrinkage, the horizontal arches will be under tension and the foregoing theory is not valid unless the arch, including its connection with the abutments, can support safely this tension, which will generally require reinforcement. Let us assume such reinforcement; then, from Fig. 18 (c), as noted, $F\,E = c\,R\,\theta + c'\,R$, and the radial unit stress, P, at the base, is

$$P = \frac{E\,t}{1.56\,R_u}\,(c\,\theta + c').$$

As regards the arch, P is the resistance of the cantilever, acting outwardly, to the supposed motion of the crown down stream. As concerns the cantilever, it is the pressure of the arch on the cantilever acting inwardly. After assuming a value of P at the crest and sketching a likely P curve, the deflections of the cantilever are found as in Fig. 19. The elastic deflections, u, of the horizontal arches, at the crown of each, for the forces, P, are then computed, say, from Equation 13 (a); whence the actual deflection of any arch is

$$u_0 = c\,R\,\theta + c'\,R - u = (c\,\theta + c')\,R - u,$$

and this should agree with the cantilever deflection at the corresponding crown. If this does not obtain for every arch, very nearly, a second trial must be made, and so on. From a final satisfactory result, the values of moments and deflections are as last computed, and the value of any circumferential tension in a horizontal arch is found from the formula, $T = P\,R_u$, or by the more accurate one already given.

The writer is unable to agree with Mr. Noetzli in his treatment of temperature changes and shrinkage. In Figs. 5 and 6, the author assumes the deflection of a horizontal arch at the crest as $1.56\, c\, R\, \theta$, whereas, it is $c\, R\, \theta - H\, L$, Fig. 18 (*c*), or $G\, H$. As seen from Table 9 and the foregoing, for the Wooling Dam, $G\, L =$ 0.178 in., $G\, H =$ 0.168 in., so that the difference is small; but, even if it is disregarded, it was clearly shown that in the supposed first "cylinder" expansion, the deflection, $G\, L$, was $c\, R\, \theta$ and not $1.56\, c\, R\, \theta$. Observe, too, in Fig. 18 (*c*) that the curve, $F\, H$, is far from straight, so that, for a uniform change of temperature, a linear variation in the deflection is inadmissible. How this may be, on the author's hypothesis, that the change of temperature in the dam varies linearly from a maximum at the crest to zero at the base, it is impossible to say. In fact, no theory exists for such a case; and, besides, it is a question of fact, to be determined by experiment, what the variation is. For a thin dam, the hypothesis of the same change in temperature throughout—at least down to the ground surface, or a few feet below—seems the more reasonable of the two hypotheses. For a thick dam, the variation is non-uniform, but until a satisfactory analysis of such a case is effected, the old hypothesis of an average uniform change may still have to be used, although admittedly inexact.

For any curved dam, the values of thrusts, moments and deflections are numerically the same for a fall of temperature as for a rise of the same number of degrees, but they will have opposite signs. It will prove instructive, after combining the numerical results for the Wooling Dam, as given in Mr. Smith's Tables 4 and 6, with their proper signs for a rise or fall of temperature of 20° Fahr. and the full water pressure, to compute the unit pressures on the cantilever and compare results. This is done in Fig. 20, where the unit pressures on the cantilever, at different depths, are marked on the horizontal ordinates representing them, as follows:

Fig. 20 (*a*), for water pressure only;
Fig. 20 (*b*), for water pressure and a rise of temperature of 20° Fahr.
Fig. 20 (*c*), for water pressure and a fall of temperature of 20° Fahr.

The pressure on the base alone can be computed independently. This pressure, for water pressure only, is 2 062 lb. per sq. ft., and for a change of temperature of 20° Fahr. is 688 lb. per sq. ft., as found previously, hence, in Fig. 20 (*b*), the pressure at the base is $2\,062 - 688 = 1\,374$ lb. per sq. ft., and in Fig. 20 (*c*) is $2\,062 + 688 = 2\,750$ lb. per sq. ft., since, for a rise of temperature only, for any d, the pressure on the cantilever is up stream in direction, and the reverse for a fall in temperature.

In the writer's discussion of Mr. Smith's paper, the hypothesis that the arches were free to move led to results differing from 0 to 5 to 10% from the computed ones. The writer has since found that the differences were largely due to the approximations introduced relative to the values of Q. In view of the remarkable agreement shown in Table 9 in the results corresponding to the two hypotheses of shear or no shear between the horizontal arches, in the case of a temperature change only, it was decided to make a similar test for the case, Fig. 20 (*c*), of combined water pressure and a temperature drop of 20° Fahr.

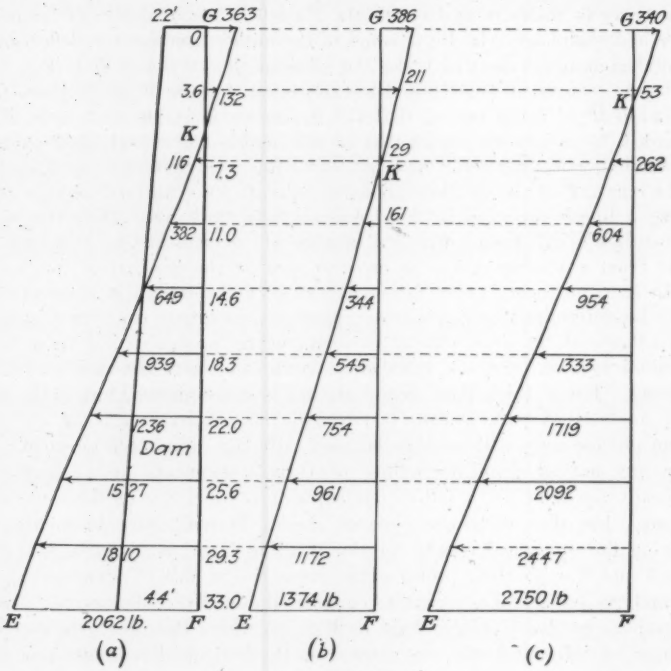


FIG. 20.

The computation, using the pressures marked on Fig. 20 (*c*), was made with care, measuring distances, as arms, to a hundredth of a foot, computing moments at nineteen points, and using a scale of distance of 3 ft. to 1 in. The results, given in Table 10, are again remarkable, showing that the two hypotheses, shear or no shear between the horizontal arches, lead practically to the same thrusts, moments, and deflections.

TABLE 10.—WOOLING DAM, RESERVOIR FULL, AND 20° FAHR. FALL OF TEMPERATURE.

d, in feet.	Moment on cantilever, in foot-pounds.	*M* (Smith), in foot-pounds.	Deflection of cantilever, in inches.	Deflection (Smith), in inches.
0.0	0	0	0.356	0.356
3.6	— 1 600	— 1 600	0.308	0.309
7.3	— 3 800	— 3 900	0.261	0.261
11.0	— 2 500	— 2 500	0.212	0.211
14.6	+ 6 800	+ 7 300	0.164	0.162
18 3	+ 29 200	+ 29 100	0.116	0.116
22.0	+ 70 000	+ 70 000	0.072	0.071
25.6	+ 132 100	+ 132 800	0.036	0.034
29.3	+ 224 200	+ 224 200	0.010	0.010
33.0	+ 349 800	+ 348 600	0.000	0.000

The thrusts, T, are not included, as they were assumed to be the same on either hypothesis. The value of T from Mr. Smith's Table 4 was combined with that from Table 6 with the sign changed, to find a resultant, T, for water pressure and fall of temperature. Then, as before, the unit radial pressure on a horizontal arch is $p = \frac{T}{135}$ and the unit pressure on the cantilever at depth, d, is $62.5\ d - p$. The same results can be obtained by adding the unit pressures on the cantilever, as given in Tables 5 and 8 of the foregoing.

If T represents the combined thrust due to water pressure and θ degrees change of temperature for any arch, it can be shown from what precedes, that the deflection of the horizontal arch, in feet, for water pressure and a rise in temperature, is,

$$D = \frac{3}{2}\frac{R}{Et}T - c\,R\,\theta,$$

and the deflection, for water pressure and a fall of temperature, is,

$$D = \frac{3}{2}\frac{R}{Et}T + c\,R\,\theta.$$

These formulas are needed in case the tentative method is applied to the combination of water pressure and a change in temperature, as illustrated in Figs. 20 (b) and 20 (c) for the Wooling Dam.

In fact, if the tentative method gives the pressures shown in Figs. 20 (a) and 20 (b), for water pressure only and combined water pressure and rise of temperature of 20° Fahr., then the difference of corresponding pressures will give the pressure on the cantilever for a temperature rise alone. The construction of Fig. 19 may now be omitted.

The aim of the writer has been to show that the hypothesis on which the author's method is based—that the horizontal arches are free to move over each other—is sound from a practical standpoint and leads to very close approximations. Then, the numerical results for the Wooling Dam were used to throw light on the author's analysis and to give confidence in the results concerning the influence of water pressure. The attempt was also made to make clear the influence of temperature changes—at least for uniform changes.

The author has done a great service in developing so simple and practical a method for dealing with curved dams and in calling attention to the probability or certainty of curved dams failing in gravity action, with the accompaniment of horizontal cracks. Apparently, the best way to guard against cracks, either vertical or horizontal, is to reinforce in both directions, including tying the dam to the base and to the abutments with sufficient reinforcement. Where this is not done, the dam is no longer "fixed" at the base, and the method developed in the foregoing is no longer valid. Mr. Smith, in the paper cited, has given a complete analysis for this case, but without a numerical illustration. As this case—where the curved dam is simply supported on the foundation and not tied to it—frequently occurs, it was thought that it would be interesting and instructive to present a completely worked-out example. With that end in view, the Wooling Dam was again selected, so that the results could be compared with those given for the same dam supposed to be fixed at the base.

For the dam not fixed at the base, but tied to the abutments where needed, the conditions to be fulfilled are: The moment and shear at the crest shall equal zero,

and the moment and displacement at the base shall likewise equal zero. From the four equations resulting, the values of A', B', etc., are found, and also the resulting displacements, arch thrusts, T, and moments, M, exactly as explained by Mr. Smith.* To do this quickly and accurately, an eight-figure calculating machine is desirable, since in the course of the elimination, very few figures are left. As the writer did not have such a machine he resorted to seven-place logarithmic tables, being careful to retain seven significant figures throughout, as far as possible, so that it is thought that the values of the displacements (or deflections), thrusts, and moments, are correct to two significant figures and generally to three, which answers the object in view.

The results are given in Table 11. The radial pressures, $p = \frac{T}{135}$, on the arches are computed from the values of T; then the resulting unit pressures on the cantilever $= 62.5d - p$. From the latter pressures the moments, m, are found, as hitherto explained, and, although approximate, they afford a useful check on the values of M, and approximately check the entire work. The value of m at the base, as computed, was — 1 000 ft-lb., whereas, it should be zero, as inserted in Table 11. There was likewise a slight irregularity in the displacements at $d = 22.0$, 25.6, and 29.3, so that the recorded values are adjusted, fitting a smooth curve.

TABLE 11.—WOOLING DAM, NOT FIXED AT BASE, RESERVOIR FULL, NO CHANGE OF TEMPERATURE.

d, in feet.	Displacement, in inches.	Arch thrust, T, in pounds.	$p = \frac{T}{135}$, in pounds per square foot.	Pressure on cantilever, in pounds per square foot.	Moment, M, in foot-pounds.	Moment, m, in foot-pounds.
0	+ 0.362	+ 94 600	701	— 701	— 0	0
3.6	+ 0.342	+ 99 100	734	— 509	— 5 700	— 4 140
7.3	+ 0.320	+ 101 900	755	— 299	— 15 400	— 15 450
11.0	+ 0.293	+ 101 900	755	— 68	— 30 250	— 30 590
14.6	+ 0.261	+ 98 300	728	+ 184	— 45 600	— 45 070
18.3	+ 0.221	+ 89 700	664	+ 480	— 59 230	— 59 370
22.0	+ 0.182	+ 79 000	585	+ 790	— 66 100	— 66 070
25.6	+ 0.130	+ 60 500	448	+ 1 152	— 61 570	— 62 140
29.3	+ 0.075	+ 36 900	273	+ 1 558	— 42 810	— 42 380
33.0	0.000	0	0	+ 2 062	0	0

NOTE.—Displacement down stream is positive, up stream is negative; when a horizontal arch is under compression, T is positive, when under tension, T is negative; the moment, M, is regarded as positive when the neutral line of the cantilever is concave down stream.

By the use of eight-place logarithmic tables, the results should be more accurate, provided eight significant figures are retained when feasible, even if the corresponding coefficient has to be written to ten or eleven decimal places; since, if an equation has to be multiplied or divided by such a number, the results are only true to eight significant figures provided such a number is retained in the multiplier or divisor.

The curves of moments, arch thrusts, and deflections are shown in Fig. 21, the vertical line representing the vertical face of the dam. The water in the reservoir is on the right of the dam, and, since the displacements or deflections are all down stream, the deflection curve is laid off to the left of the vertical, although the

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 2064–2075.

deflections are given the positive sign in Table 11. The values of T, being everywhere positive (indicating compression), are laid off to the right of the vertical, and the values of M, being negative throughout, are laid off to its left. The curves are very different from those pertaining to the case previously considered of the dam fixed (or encastrée) at the base, the arch thrusts and deflections being more than twice as great, on an average, and the moments on the cantilever, being greater, except from $d = 25.6$ ft. to the base, where the moment is zero. It will be observed that the tangent to the deflection curve at the base is inclined down stream and that the deflection curve bends to the right, or is concave up stream, which corresponds to the negative values of the moments for the whole height of the dam.* The tentative solution, which applied for the case of the dam fixed at the base, does not apply here, since the inclination of the tangent to the deflection curve at the base is unknown. As might be expected, since the cantilever not fixed at the base cannot offer as much resistance as one fixed at the base, the

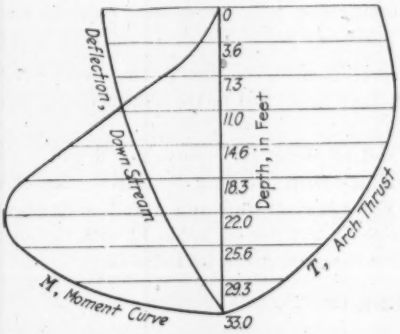


FIG. 21.

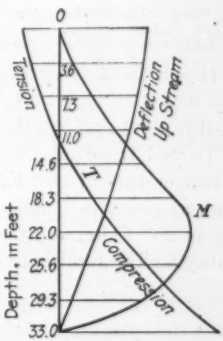


FIG. 22.

horizontal arches have to sustain a much greater load and the deflections are correspondingly greater than for the case of the dam fixed at the base.

In Table 12 the results, for reservoir empty, but for a temperature rise of 20° Fahr., are given. Here, since there is no water pressure, the radial pressure, p, is the unit pressure on the horizontal arches, and minus this is the reaction of the arches on the cantilever, from which the moments, m, are computed as usual.

The value of m at $d = 33.0$ (the base), as computed, was — 1 000, in place of the true value, 0, as inserted in Table 12. The curves of displacements, thrusts, and moments are smooth, so no adjustments were made. They are shown in Fig. 22; the deflections, being up stream, are laid off to the right of the vertical line, representing the vertical face of the dam. The tangent line to the deflection curve at the base makes an angle of nearly 3 min. to the right of the vertical. From this inclined tangent the cantilever bends to the left, the deflection curve being

* The original vertical generator at the base is now inclined there to the left of the vertical. This is due entirely to the difference in the arch thrusts at and just above the base. Thus, at the base, $T = 0$, and there is no displacement: whereas, at $d = 29.3$, $T = 36\,900$ lb. compression, resulting in a shortening of the horizontal arch there and a consequent displacement down stream. There is no vertical stress on the base from water pressure alone. Such vertical pressure comes from the weight of the dam, which has to be ultimately considered.

concave down stream, which agrees with the positive signs of the moments. The scale of the M curve is larger than that in Fig. 21, in order to give a plainer figure.

TABLE 12.—WOOLING DAM, NOT FIXED AT BASE, RESERVOIR EMPTY, 20° FAHR. RISE OF TEMPERATURE.

d, in feet.	Displacement, in inches.	Arch thrust, T, in pounds.	$p = \frac{T}{135}$, in pounds per square foot.	Pressure on cantilever, in pounds per square foot.	Moment, M, in foot-pounds.	Moment, m, in foot-pounds.
0	— 0.270	— 23 120	— 171	+ 171	0	0
3.6	— 0.245	— 18 630	— 138	+ 138	+ 2 100	+ 1 084
7.3	— 0.220	— 12 700	— 94	+ 94	+ 4 000	+ 3 940
11.0	— 0.194	— 4 930	— 37	+ 37	+ 8 200	+ 8 100
14.6	— 0.167	+ 4 960	+ 37	— 37	+ 12 850	+ 12 600
18.3	— 0.137	+ 17 280	+ 128	— 128	+ 17 010	+ 16 720
22.0	— 0.105	+ 32 350	+ 240	— 240	+ 19 290	+ 19 020
25.6	— 0.072	+ 49 740	+ 368	— 368	+ 18 590	+ 18 080
29.3	— 0.036	+ 70 080	+ 519	— 519	+ 12 750	+ 12 070
33.0	— 0.000	+ 92 880	+ 688	— 688	0	0

NOTE.—The displacement, being negative, is up stream; when T is negative, the arches are in tension, and in compression when T is positive; positive M indicates that the deflection curve of the cantilever is concave down stream.

It will be observed that T is negative (indicating tension in the arches) from $d = 0$ to $d = 12.8$, so that the dam will have to be tied to the abutments, at least for this depth, for the theory to be valid.

To find the displacements, thrusts, and moments for reservoir full and a rise of temperature of 20° Fahr., simply add the corresponding quantities with their signs as given in Tables 11 and 12. For reservoir full and a fall of temperature of 20° Fahr., add the results of Table 11 to those of Table 12 with the signs changed. The resulting values for the two cases are given in Table 13.

TABLE 13.

d, in feet.	RESERVOIR FULL, RISE OF TEMPERATURE, 20° FAHR.			RESERVOIR FULL, FALL OF TEMPERATURE, 20° FAHR.		
	Deflection, in inches.	T, in pounds.	M, in foot-pounds.	Deflection, in inches.	T, in pounds.	M, in foot-pounds.
0	+ 0.092	+ 71 480	0	+ 0.632	+ 117 720	0
3.6	+ 0.097	+ 80 470	— 3 600	+ 0.587	+ 117 730	— 7 800
7.3	+ 0.100	+ 89 200	— 11 400	+ 0.540	+ 114 600	— 19 400
11.0	+ 0.099	+ 96 970	— 22 050	+ 0.487	+ 106 880	— 38 450
14.6	+ 0.094	+ 103 260	— 32 750	+ 0.428	+ 93 340	— 58 450
18.3	+ 0.084	+ 106 980	— 42 220	+ 0.358	+ 72 470	— 76 240
22.0	+ 0.077	+ 111 350	— 46 810	+ 0.287	+ 46 650	— 85 390
25.6	+ 0.058	+ 110 240	— 42 980	+ 0.202	+ 10 760	— 80 160
29.3	+ 0.039	+ 106 980	— 30 060	+ 0.111	— 33 180	— 55 560
33.0	0.000	+ 92 880	0	0.000	— 92 880	0

Although the figures in Table 13 are given as computed, they can only be relied on to two or three significant figures, owing to the use of only a seven-place logarithmic table. It will be observed, for the case of reservoir full and a fall of temperature of 20° Fahr., that tension exists in the arches near the base. Also, from Table 12, for the case of reservoir empty, a rise of temperature gives T minus, or tension, for the upper arches and a fall in temperature causes tension in the lower arches, so that, to meet every condition, the dam must be tied to the abut-

ments for its entire height with proper reinforcement; otherwise, the analysis is not valid except for the case of reservoir full (Table 11), or of reservoir full and a temperature rise (Table 13), since there is no tension in the arches in either case

The author has discussed the case where the dam is neither fixed to the base nor to the side-walls or abutments under the heading "Curved Dams as Cylinder Hoofs", but the solution pertains only to the cantilever and does not include arch action, so far as finding arch thrusts are concerned. It would seem, then, in order to assure definite results, that the dam should be securely tied to the abutments, in which case the analysis of this discussion is valid. Of course where the dam is not tied to the base, the upper part of the joint may open and admit water, with its uplift, so as to change the conditions materially. The writer therefore believes that curved dams should be designed and built with sufficient vertical and horizontal reinforcement to ensure "fixity" at the base and abutments and to prevent any cracks in any direction.

Mr. Smith's analysis dealt with the case where the dam is vertical on the water side. This analysis would seem to be equally true when the water face is battered, except that the weight of water vertically over this face must be combined with the weight of the dam. The dam and this water over it are to be considered as without weight in the primary analysis.

It is very fortunate that two such able papers as those by the author and Mr. Smith should have been presented about the same time, for if the established results are heeded, they should lead to better designs.

B. F. JAKOBSEN,* ASSOC. M. AM. SOC. C. E. (by letter).—The author has followed the method outlined and applied by Silas H. Woodard,† M. Am. Soc. C. E., in his calculations for the Lake Cheesman Dam, and has assumed that the division of load between the vertical cantilever and the horizontal arches can be computed from the respective deflections of the cantilever and the horizontal arches. There is no doubt regarding this, for it is evident that load which is not taken by the cantilever must be taken by the arches. The only question involved is how these deflections should be calculated.

From a consideration of the cantilever and arch deflections, not as these really are, but as his calculations show they ought to be, the author condemns curved dams having sections less than gravity dams and insists that these dams must have failed in bending before the arches could deflect sufficiently to sustain the load. The author cites no facts which would bear him out; in fact, he seems to be quite well aware that so far as is known, none of the things which he claims should happen has actually taken place. He has made certain assumptions, which it is convenient to make in order to facilitate mathematical treatment, and has then mistaken his assumptions for facts. The conclusions which he draws show a lack of understanding of theoretical application, as well as a lack of knowledge regarding the manner in which materials like concrete act under stress.

Regarding the "scientific design" of dams, on which he insists, it should have been evident to the author that even if a dam could be designed so that the deflections due to cantilever and arch are made to coincide reasonably well at

* Fresno, Cal.

† *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 108.

the crown and at a given temperature, they would not coincide at some other vertical plane, nor at some other temperature. The stress distribution in curved dams is vastly more complicated than the author's calculations would lead one to suspect.

In order to justify his sweeping and general conclusions, which are apparently greatly at variance with facts, the author should have submitted a theory which is complete and which includes all the facts. It is the purpose of the following discussion to show that the author's theoretical deductions are as incorrect as his conclusions.

Theory cannot agree with facts except when the assumptions on which it is founded agree with the facts in the case. The general theory of flexure requires that

1.—A force acting to bend a bar must act perpendicularly on the neutral axis of the bar;
2.—The modulus of elasticity is constant both for compression and for tension;
3.—The modulus of elasticity is the same for compression as it is for tension;
4.—The deformation due to shear is negligible;
5.—The influence of lateral deformation (Poisson's ratio) is negligible.

This last requirement is especially important when a body is stressed in several directions, as is the material in a curved dam. To these the author has added two more, namely, that

6.—The foundation is unyielding; and,
7.—The water soaking the up-stream face does not increase the volume.

None of these requirements is even approximately complied with in this case, and the actual cantilever deflections are so much greater than the calculated values that one need not be astonished when the author (page 54) finds the theoretical cantilever deflection of the Lake Spaulding Dam to be only about one-tenth of the actual deflection. From a rough estimate of the deflection at the top due to yielding of the foundation, the writer believes this deflection alone would be nearly as great as the total deflection which the author gives.

Requirement No. 1.—From this requirement, that a force acting to bend a bar must act perpendicularly on the neutral axis, it follows that the center of curvature of this axis must lie somewhat as shown in Fig. 23. The force, P_h, due to the water pressure, should be resolved into one force acting perpendicularly on the neutral axis, and another force acting parallel to this axis and tending to compress the dam.

Since the theory of flexure is based on a prismatic bar, it may be well to see how closely it will apply to a triangular section. For this purpose, the stresses in a gravity dam, for reservoir empty, are shown in Fig. 24. Any small elementary body inside the dam is evidently under compression due to the weight of the concrete lying above it. At the up-stream face an elementary body lying h ft. below the water surface is stressed by a force, c_h, and if w is the weight of 1 cu. ft. of concrete,

$$c_h = w\ h \qquad \text{.....(37)}$$

in pounds per square foot.

The proportion between the height, h, and the base, b, at any point, must be such that when the reservoir is full, $c_h = 0$, that is, no tension must exist in the up-stream face of the gravity dam.

When the reservoir is full, the water tends to bend the cantilever and this produces tension in the up-stream face of the dam. This may be computed as follows:

The force acting on a vertical slice of dam 1 ft. wide, h ft. below the water surface, is:

$$P_h = 0.5\, q\, h^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

in pounds per square foot.

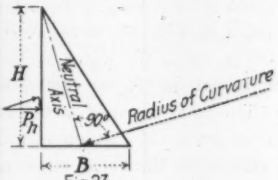
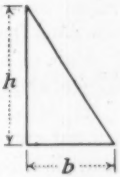
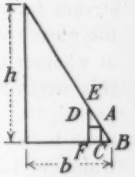


Fig 23





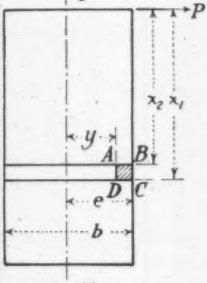


Fig 26.

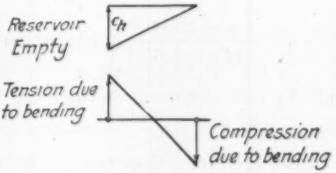


Fig 24 Bending Diagram

Fig 25

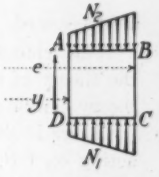


Fig 27

The lever arm of this force is $\frac{h}{3}$ ft. and the moment is

$$M_h = \frac{q}{6}\, h^3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (39)$$

in foot-pounds, where q is the weight of a cubic foot of water. The section modulus is $\frac{b^2}{6}$, and the stress due to bending at the extreme fiber (at the up-stream face), is:

$$t_h = \frac{q\, h^3}{b^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (40)$$

in pounds per square foot.

Since this must be equal to c_h,

$$w\, h = \frac{q\, h^3}{b^2}$$

$$\frac{h}{b} = \sqrt{\frac{w}{q}} = a \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (41)$$

where a^2 is the ratio of the weight of the concrete to that of water. If concrete weighs 150 lb. per cu. ft. and water 62.5 lb., $a^2 = 2.4$ and $a = 1.55$; then $b = 0.667\,h$, or the base is two-thirds of the height.

By combining the stress diagram for reservoir empty, Fig. 24, with the bending diagram, the stress diagram for reservoir full is obtained, as shown in Fig. 25. The full line shows the stresses with reservoir full, and the dash line shows the stresses for reservoir empty.

Even a cursory examination will show that this stress diagram, on which the author bases his calculations of deflection for the cantilever, cannot represent the actual stresses in a gravity dam.

The beam theory demands that the shear stress at the up-stream and down-stream faces be zero. The maximum compressive stress at the base h ft. below the water surface is at the toe of the dam, that is, at point B, Fig. 25. Irrespective of whether the dam is only 200 ft. high, or higher, 200 ft. below the water surface this stress is 200×150, for concrete weighing 150 lb. per cu. ft., or 30 000 lb. per sq. ft. If BC in Fig. 25 is 1 ft., AC is 1.55 ft. The total force of 30 000 lb. must be transferred toward the neutral axis through the area, AC, which is 1 ft. wide and 1.55 ft. high. If the shear distribution is parabolic, the shear stress on AC is

$$s = \frac{1.5 \times 30\,000}{144 \times 1.55} = 202 \text{ lb. per sq. in.}$$

This, however, should be zero, according to the beam theory, provided $\frac{B\,C}{b}$ is small, as is assumed.

The triangular beam, therefore, does not act as the prismatic beam on which the theory of flexure is based, and certain modifications are necessary before this theory can be applied to a triangular section.

There is also tension at C and compression at A. The moment of the force, c_h, acting on CB, is $0.5 \times 30\,000 = 15\,000$, and as $AC = 1.55$ ft. and the section modulus, therefore, is $\frac{2.4}{6}$, the bending stress at the extreme fibers is

$$c_b = t_b = \frac{15\,000 \times 6}{2.4 \times 144} = 260 \text{ lb. per sq. in.}$$

The restraining action of the foundation would take care of some of this bending stress, but a short distance away from the foundation this restraining action is not present or, if it is present, must produce additional shear in the dam.

If any such stresses were actually present in gravity dams, this type of dam would no doubt show a respectable percentage of failures, but as far as the writer knows, no dam has ever failed due to excessive tension and shear in the toe. This fact would tend to show that no such stresses exist.

This stress-distribution problem may be approached in a somewhat different manner. At the point, B, in Fig. 25, according to the stress diagram, the force is $c_h = 208$ lb. per sq. in., and directly above B, this force is zero. It is reasonable, therefore, to assume that the force in B is actually zero or very nearly so, instead of being a maximum for that horizontal section.

The actual stress diagram on which calculations for deflections should be based is, therefore, materially different from the one on which the author has based his

calculations. The maximum stress and the whole stress diagram, therefore, must be moved toward the up-stream face and there is some tension in the up-stream face, hence the deflections of the vertical cantilever are much larger than the theory of flexure of the prismatic beam would indicate.

For comparison, it may be well to review briefly the theory of the prismatic beam. Fig. 26 shows such a beam, fixed at the lower end and stressed with a force, P, at the top. The beam will be considered as 1 ft. wide in a direction perpendicular to the plane of the paper. The compression at point D of the small square, $ACBD$, which is shown to a large scale in Fig. 27, is

$$c_y = \frac{P x_1}{I} y,$$

where I is the moment of inertia of the section of width, b. At point C the compressive stress is

$$c_e = \frac{P x_1}{I} e$$

The area of DC is $e - y$, and the compression acting on this area is

$$N_1 = \frac{c_y + c_e}{2} (e - y) = \frac{P x_1}{I} \times \frac{e^2 - y^2}{2}.$$

For the area, AB,

$$N_2 = \frac{P x_2}{I} \times \frac{e^2 - y^2}{2}$$

N_1 is greater than N_2, because x_1 is greater than x_2. The force $N_1 - N_2$, therefore, must be transmitted through the area, $AD = x_1 - x_2$. If s is the shear per unit area,

$$s (x_1 - x_2) = N_1 - N_2$$

and, consequently,

$$s = \frac{P}{I} \frac{e^2 - y^2}{2} = 1.5 \frac{P}{b} \left(1 - \frac{y^2}{\left(\frac{b}{2}\right)^2} \right)$$

This shows the shear to be distributed parabolically across the area, b, of the prismatic beam. This shear is zero for $y = \frac{b}{2}$, that is, for the extreme fibers, and it is a maximum for the neutral axis, where $y = 0$, where it is

$$s_{max.} = 1.5 \frac{P}{b}$$

Since the resultant of the forces, N_1 and N_2, lies outside of AD in Fig. 27, there must also be shear forces acting in DC and in AB. These shear forces are equal to the shear in AD, as may readily be shown.

A comparison between the stresses in the prismatic beam and those in the triangular section shows that entirely different conditions are met with, and the theory is not applicable without modification.

Requirement No. 2.—This requires that the modulus of elasticity must be constant both for compression and for tension. Bach gives* for compression tests

* "Elasticität und Festigkeit," Third Edition, p. 58.

on 1:2½:5 gravel concrete, for stresses less than the maximum safe stress to be used in construction, the deformation per unit length, as follows:

$$e = \frac{s^n}{E} \qquad (42)$$

where n varies from 1.137 to 1.207; s is the stress in pounds per square inch, and E the modulus of elasticity.

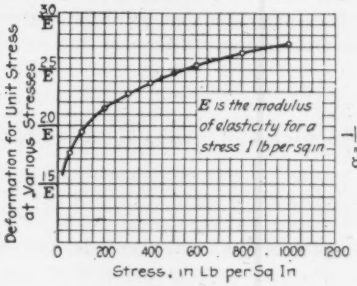


FIG. 28.

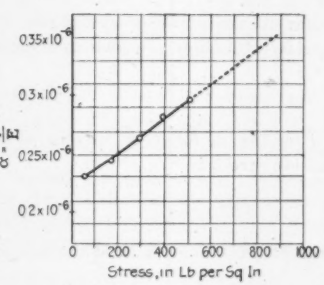


FIG. 29.

Equation (42) shows that the deformation increases much faster than the stress. For $n = 1.145$, for example, the relation between the deformation and the stress is shown in Fig. 28. The deformation when 100 lb. per sq. in. is applied, is:

$$e = 1.95 \frac{100}{E} = \frac{195}{E}$$

while for 1 000 lb. per sq. in., it is:

$$e = 2.72 \frac{1\,000}{E} = \frac{2\,720}{E}$$

or, the deformation is increased 14.2 times, while the load is increased only 10 times.

According to tests by Bach* of 1:2½:5 gravel concrete, 77 days old, the relation between stress and deformation is as given in Table 14.

TABLE 14.

Stress limits, in pounds per square inch.	Average stress, in pounds per square inch.	E	$\frac{1}{E}$
0 — 112	56	4 350 000	0.23×10^{-6}
112 — 224	168	3 640 000	0.275×10^{-6}
224 — 337	281	3 210 000	0.312×10^{-6}
337 — 450	394	3 010 000	0.332×10^{-6}
450 — 560	505	2 750 000	0.364×10^{-6}

In order to calculate the deformation due to a stress of 212 lb. per sq. in., for example, calculate first the deformation due to 112 lb. per sq. in., with $E = 4\,350\,000$. To that add the deformation which occurs when the stress is increased from 112 to 212 lb. per sq. in., with $E = 3\,640\,000$.

* Hütte "Des Ingenieurs Taschenbuch," 21st Edition (1911), Vol. 1, p. 520.

In order to plot this, Table 14 was re-calculated, as follows:

Stress, in pounds per square inch.	Coefficient of elasticity, E.
56	4 350 000
168	4 090 000
281	3 790 000
394	3 550 000
505	3 380 000

This is plotted in Fig. 29, in terms of $\alpha = \frac{1}{E}$ since this is proportional to the deformation and, therefore, shows at a glance the relation between deformation and stress, while if E was plotted, it would be necessary to estimate its reciprocal, α, in order to arrive at the deformation.*

Fig. 29 shows a rapid increase in the elasticity coefficient, $\alpha = \frac{1}{E}$. For 100 lb. per sq. in. the deformation is 23.6×10^{-6}, while for 500 lb. per sq. in., the deformation is $500 \times 0.296 \times 10^{-6} = 148 \times 10^{-6}$. The ratio of deformations is 1:6.28, while the ratio of stresses is only 1:5. The fact that in concrete deformation increases faster than stress is well established, and cannot be neglected.†

The fallacy of assuming an approximately constant modulus of elasticity for stresses in concrete up to the ultimate stress, is apparent, even if compressive stresses only are involved.

Requirement No. 3.—As to this requirement, that the modulus of elasticity is the same for compression as it is for tension, Hütte gives 0.88 to 0.73 for the ratio of the modulus of elasticity in tension to that in compression, for stresses between 45 and 85 lb. per sq. in. Therefore, on an average, the deformation due to tension is 1.25 times the deformation due to the same stress acting in compression. Fig. 549*b*, Johnson's "Materials of Construction", page 604*b*, shows for 1:2:4 stone concrete in tension $E_t = 3\,700\,000$, and in compression $E_c = 5\,500\,000$, or a ratio of these moduli of elasticity of 0.673. In this case the deformations due to tensile stresses are nearly 50% larger than those due to compressive stresses of the same magnitude.

The writer can find no tests giving the variation of the modulus of elasticity for tension with the increase in stresses, but Bach gives,‡ for the deformation per unit length of granite due to a compressive stress of s kg. per sq. cm.,

$$e = \frac{s^{1.132}}{250\,000} \qquad (43)$$

and for tension,

$$e = \frac{s^{1.374}}{235\,000} \qquad (44)$$

These are plotted in Fig. 30, and show a greater deformation and a much faster increase in the deformation due to tension than in that due to compression, and

* This is entirely aside from the fact that the definition of E for compression involves the nonsense of compressing a bar of certain length down to no length whatever, while $\alpha = \frac{1}{E}$ is the deformation in feet which a stress of 1 lb. per sq. in. produces in a bar 1 ft. long. This elasticity coefficient, α, also has an advantage in indicating when one material is more elastic than another, since α is larger in the first case than in the second, while with E the reverse is true.

† See, for example, *Technologic Paper No. 58*, U. S. Bureau of Standards (1916), Fig. 21, p. 44. Also Johnson's "Materials of Construction" (1912 Edition), Figs. 547 and 549*d*.

‡ "Elasticität und Festigkeit", Third Edition, p. 60.

it is reasonable to assume that something quite similar takes place in concrete. Therefore, the modulus of elasticity for concrete is not even approximately the same for compression as it is for tension, nor is it constant or even approximately constant for either compression or tension.

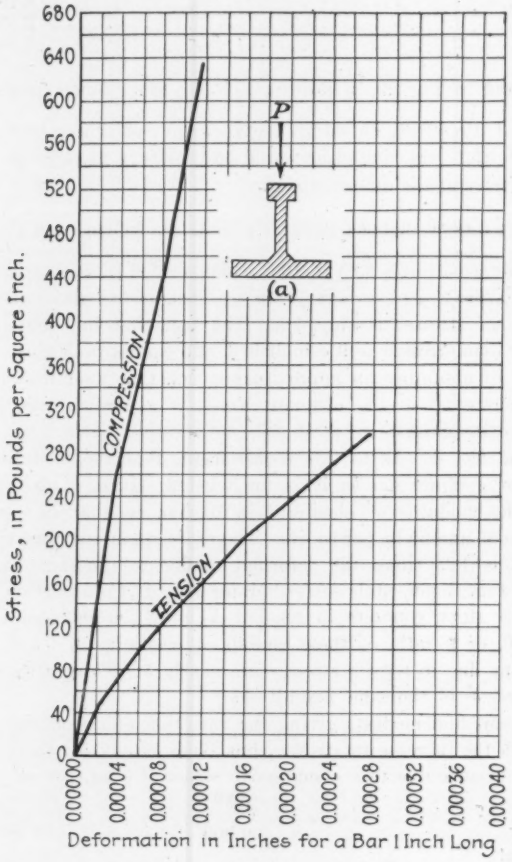


FIG. 30.

The following may serve to show the influence of the variation of the modulus of elasticity. A bar of length L in. and section as shown in Fig. 30 (*a*), supported at the two ends and loaded with a force of P lb. at the center, should deflect in the middle,

$$y = \frac{P\,L^3}{48\,E\,I}$$

where I is the moment of inertia.

It is immaterial, therefore, whether the wide or the narrow flange is stressed in tension. Actual tests show, however, that for cast-iron beams a considerably

larger deflection occurs when the narrow flange is in tension, than when the reverse is the case.*

Requirement No. 4.—This assumes that the deformation due to shear is negligible. As far as the writer knows, no one has ever taken into consideration the shear deformation in dams. The author does not mention it, and Mr. Woodard† states that the distortion caused by shear is practically zero.

In a gravity dam 200 ft. high, the deflection due to shear at a point 100 ft. above the base is practically as much as the bending deflection,‡ and 25 ft. above the base it is nearly five times as much. Still nearer the base, this shear deformation is practically the only deformation in a cantilever. This, of course, is on the assumption of unyielding foundation, which is an incorrect assumption that results in a deflection near the base much smaller than it really is.

The shear deflection may be found, as shown later,§ when the parabolic distribution of shear across the base and the consequent distortion of the horizontal planes are taken into account. If the shear is assumed as evenly distributed over the base, the shear deflection is 20% less‖ than if the parabolic distribution is taken into account. The deflection due to shear in a body of height dx ft. and depth b ft., Fig. 31, when S_h is the total shear force in pounds acting in the horizontal plane, is, in feet,

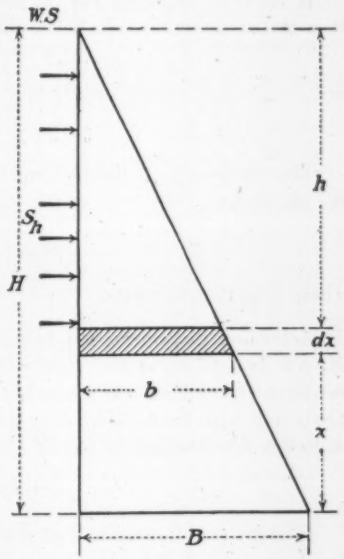


FIG. 31.

$$d\,y_h = \frac{1.2}{144\,G}\,\frac{S_h}{b}\,d\,x \qquad (45)$$

where G is the coefficient of rigidity, in pounds per square inch.

The deflection x ft. above the base is, therefore,

$$y'_h = \frac{1.2}{144\,G}\int_0^x \frac{S_h}{b}\,d\,x.$$

In a gravity dam,

$$S_h = 31.3\,h^2$$
$$= 31.3\,(H-x)^2$$

Let

$$a = \frac{h}{b} = \text{constant},$$

* Bach, "Elasticität und Festigkeit," 3d Edition, p. 202.
† *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 119.
‡ As calculated by the author.
§ Bach, "Elasticität und Festigkeit", 3d Edition, p. 420.
‖ Johnson's "Materials of Construction", 1912 Edition, p. 66.

where

$$a^2 = \frac{\text{weight of concrete}}{\text{weight of water}}$$

If concrete weighing 150 lb. per cu. ft. is assumed, $a^2 = 2.4$ and $a = 1.55$, then the deformation h ft. below the water surface is, in feet,

$$y'_h = \frac{1.2}{144\ G} \times 31.3 \int_0^x \frac{(H-x)^2}{(H-x)}\, a\, d\, x$$

$$= 0.261\ \frac{a}{G} \left(H\, x - \frac{x^2}{2} \right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (46)$$

The coefficient of rigidity and modulus of elasticity are connected through the equation:

$$\frac{1}{G} = 2\ \frac{m+1}{m} \times \frac{1}{E} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (47)$$

where $\frac{1}{m}$ is Poisson's ratio.

According to information furnished by Professor Bach to L. R. Jorgensen,* M. Am. Soc. C. E., $m = 5$ for concrete, so that $E = 2.4\ G$. Hütte† gives for the coefficient of rigidity for concrete the values, 1 140 000 to 1 280 000, which agree fairly well with the relation, $E = 2.4\ G$. Introducing this into the formula for the deflection due to shear, in feet,

$$y'_h = 0.626\ \frac{a}{E} \left(H\, x - \frac{x^2}{2} \right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (48)$$

In order to obtain an idea of the deflections due to bending and shear, Table 15 has been computed for a gravity dam 200 ft. high and a modulus of elasticity, $E = 3\,210\,000$.

TABLE 15.

h, in feet.	x, in feet.	y_h, in feet.	y'_h, in feet.	$y_h + y'_h$	$\frac{y_h + y'_h}{y_h}$
0	200	0.02012	0.00608	0.02620	1.30
25	175	0.01540	0.00600	0.02140	1.39
50	150	0.01133	0.00570	0.01703	1.50
75	125	0.00787	0.00523	0.01310	1.53
100	100	0.00503	0.00456	0.00959	1.91
125	75	0.00284	0.00371	0.00655	2.31
150	50	0.00126	0.00266	0.00392	3.11
175	25	0.00031	0.00143	0.00174	5.62
200	0	.....	.....	.....	....

These values have been plotted in Fig. 32. It is to be remembered that these deflections are less than the actual deflections, due to the facts already stated, and, also, because E is a variable, unyielding foundations were assumed and lateral deformation was neglected. The figures in Table 15 are only roughly relative values.

Requirement No. 5.—This assumes that the influence of lateral deformation is negligible. When the reservoir is empty, an element of the up-stream face of the dam is under compression, due to the weight of concrete above. As the water

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 905.

† "Des Ingenieurs Taschenbuch," 21st Edition.

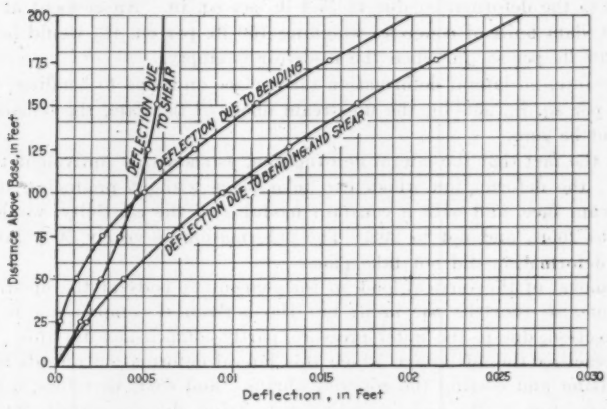


FIG. 32.

pours into the reservoir and rises in front of the dam, the cantilever is bent down stream and this decreases the vertical compression on the element of the up-stream face. (See Figs. 33 and 34.) If the dam is a gravity dam and the resultant of the triangular dam section goes to the middle third, the vertical compression in the up-stream face of the dam becomes zero. If the dam is curved, there may still be vertical compression in the up-stream face, depending on the division of load between the cantilever and the arch.

The elementary body of the up-stream face of an arch dam, when the reservoir is full, is under compression in a horizontal plane and in a down-stream direction, due to the water acting on the up-stream face (Fig. 34), and is also under compression in a horizontal plane and in a direction parallel to the up-stream face, due to horizontal arch action. Both these compressions produce an elongation in a vertical direction.

For a 200-ft. dam, the pressure due to water at the base is 87 lb. per sq. in., and with Poisson's ratio of 0.2 for concrete, the vertical elongation due to this water pressure is the same as though a force of $87 \times 0.2 = 17.4$ lb. per sq. in., acted vertically in tension.

If, in addition, an arch stress of, say, 200 lb. per sq. in., is present, this will produce an additional elongation in a vertical direction equivalent to that due to 40 lb. per sq. in. acting vertically in tension. This makes a total elongation

FIG. 33.

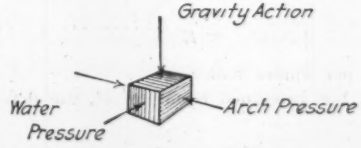


FIG. 34.

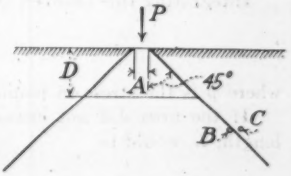


FIG. 35.

equivalent to the deformation due to 57.4 lb. per sq. in. An element at the base of a 200-ft. dam built of concrete, weighing 150 lb. per cu. ft., would be under a stress of 208 lb. per sq. in. when the reservoir is empty.

If, therefore, no lateral deformation takes place and, due to bending, a tension of 208 lb. per sq. in. acts on the up-stream fibers of the dam, the tension at this place would be zero.

Due to the fact that the lateral deformation, as shown, is equivalent to 57.4 lb. per sq. in., the deformation which can take place without producing tension in the up-stream face, and with a constant modulus of the elasticity, would be proportional to 265.4, and not to 208. The deformation is nearly 30% larger than if lateral deformation did not take place.

The removal of the vertical load on the elementary body of the up-stream face of the dam also shortens the arch; but the horizontal compression in a down-stream direction, due to the water pressure, partly compensates for this.

Counteracting the advantage which this lateral deformation affords is the fact that in setting and cooling the concrete shrinks, and that, therefore, a somewhat larger deformation of the arch is necessary to bring about a certain stress. (See Requirement No. 7 following.)

Requirement No. 6.—Concerning this requirement, that the foundation be unyielding, the writer knows of no tests which bear directly on the subject, but since a particle belonging to the rock foundation and located, say, 1 in. from the joint between the concrete and the foundation, is subject to the same stress as the concrete, and is under nearly the same pressure conditions as regards lateral constraint, it is perfectly obvious that the foundation will yield. In order to obtain a rough idea of the magnitude, assume that a uniformly distributed force, P lb., acts on the foundation, over an area, A ft. square, Fig. 35. Assume that the force is transmitted uniformly at an angle of 45°, although this, of course, is not correct, since the stress at point B cannot be considerable and the stress at C is zero, for example, but the stress will no doubt be larger the nearer the center of the pyramid is approached. For an approximate estimate, however, these assumptions may be accepted.

The area D ft. from the surface is $(A + 2D)^2$, and the stress is:

$$\frac{P}{(A + 2D)^2}$$

The deformation of an elementary body of area $(A + 2D)^2$ and thickness dD is

$$d e = \frac{P\, d\, D}{E(A + 2\, D)^2}$$

Integrating this between $D = 0$, and $D =$ infinity, gives for the deformation, e,

$$e = \frac{P}{2\, A\, E} = \frac{p\, A}{2\, E} \qquad (49)$$

where p is the stress in pounds per square foot.

If the area did not change, but remained equal to A^2, the deformation in a length, L, would be

$$e' = \frac{P\, L}{E\, A^2} \qquad (50)$$

If $e = e'$, then $L = \frac{A}{2}$. In other words, the foundation would yield as if it was of constant area, A^2, and $\frac{A}{2}$ ft. long.

Since the length, A, in dams is considerable, the foundation deformation will also be considerable. The foundation of a dam, of course, will also yield due to shear, and therefore the assumption that the arch cannot act near the bottom is erroneous. The abutments of the arch, of course, will also yield. Near the foundation many dam sites are very narrow, and when the thickness of the arch is approximately equal to its length, the dam will probably act as a wedge, as pointed out by M. Delocre.*

Since most high dams will carry a considerable part of the load near the base in cantilever action, it is evident that the assumption of unyielding foundation gives a cantilever deformation near the base which is much too small. The author is not satisfied to neglect the deflection due to the deformation of the foundation, but (page 54) computes the deflection of the Lake Spaulding Dam, of which the lowest 60 ft. are of gravity section, by assuming the cantilever to start at the top of the 60-ft. gravity section, as though entirely unaware of the fact that even if the deflection of the 60 ft. of gravity section (both the bending and shear deflection) is allowed for, the deflection thus arrived at would still be too small. He finds for the triangular cantilever a deflection of 0.36 in., and says:

"This value (compared with the deflection as actually measured, or 3.44 in.) shows that the cantilever deflected, as measured, about ten times as much as the laws of elasticity really would permit. This is possible only if the cantilever broke; and it appears there must have opened, on the water side of the dam, horizontal cracks of considerable size, aggregating probably not less than ½ in. at the up-stream face. After the cantilever had been broken, arch action could develop and carry the water pressure."

It is evident that no such conclusions are warranted.

Requirement No. 7.—In connection with this requirement, that the water soaking the up-stream face of the dam does not increase the volume, some tests made by F. R. McMillan,† M. Am. Soc. C. E., in order to determine the shrinkage of concrete in setting, show the influence of water soaking on 1:2:4 limestone concrete beams, 4 by 5 in. by 48 in. long. For instance, one of these beams, 180 days old, after having shrunk 0.0005 in. per 1-in. length, was placed in water and immediately stretched 0.00028 in. per 1-in. length, due to water soaking. This test also shows that the beam which was kept under water for 80 days, did not shrink at all due to setting. Mr. McMillan's Fig. 13 is reproduced herewith as Fig. 36.

It is evident that the effect of water soaking of the up-stream face is far from being negligible. Its effect is, in fact, to throw proportionately more load on the up-stream part of the arch, and, therefore, to make the arch stresses more nearly uniform. If the up-stream part of the arch did not swell, it is likely that the arch stresses would be a minimum at the up-stream face and a maximum at the down-stream face.

* *Annales des Ponts et Chaussées*, 4e Serie (1866), p. 212.

† University of Minnesota "Studies in Engineering No. 3," on "Shrinkage and Time Effects in Reinforced Concrete."

General Remarks.—The writer has shown in the foregoing why the cantilever deformation of the dam must be much larger, and most likely several times larger, than the simple equation for bending would indicate. It is very unlikely, therefore, that any breaking of the concrete would take place in the lower part of a high arch dam. One additional reason is that any material, when subjected to large direct pressures in all directions, and with the shear approximately constant along any plane, becomes more or less plastic and is able to undergo much larger deformations without having its physical properties destroyed than would be the case if the material were stressed in one direction only.*

The yielding in shear of a material stressed in all directions may be conceived as similar in its nature to what happens to glaciers.† Mohr reports tests by Föppl

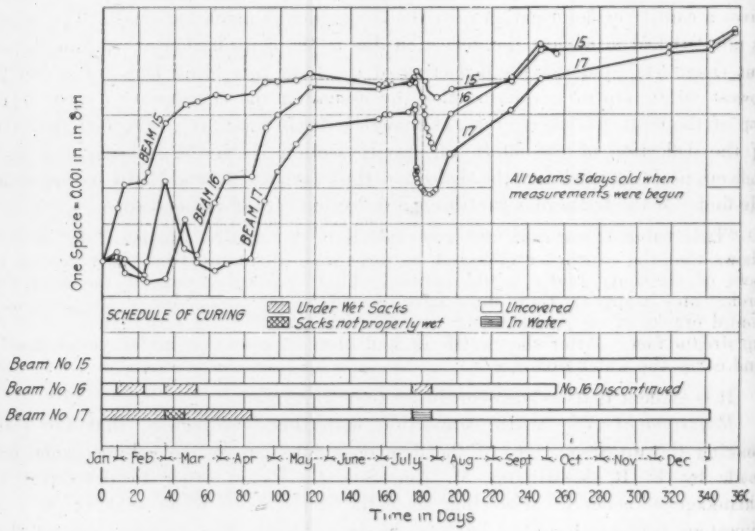


FIG. 36.

in which concrete was broken by immersing it in water and subjecting it to high pressure, but this was probably due to the fact that the porosity was not the same in all directions.

Regarding temperature stresses in the lower part of high curved dams, the temperature variation is small, especially at the up-stream face, and the effect of lateral deformation due to the weight of the overlying concrete, will, in most cases, more than balance any temperature shrinkage likely to take place.

It is a fact that temperature cracks do occur near the top in high curved dams. This can be and should be provided for by closing the dam at as low a temperature as possible and, if necessary, by forcing grout into vertical contraction joints. The

* Otto Mohr, "Abhandlungen aus dem Gebiete der Technischen Mechanik," p. 206.

† L. V. Pirsson and Charles Schuchert, "Text Book of Geology", 1915 Edition, pp. 120-122.

time effect may be relied on to re-distribute the stresses in case any unbalanced stresses should be caused.*

In curved dams these vertical temperature cracks near the top are not a source of weakness, but merely result in a small additional deformation of the arch and the cantilever.

Deflection of Horizontal Arch.—The formula† given by the late R. Shirreffs, M. Am. Soc. C. E., appears more nearly correct than the one derived by the author; it gives the deflections for large central angles practically the same as the author's formula, but for small central angles the author's formula gives values about twice as great as Mr. Shirreffs' formula, which latter takes account of the restraining action of the shorter arches. The crown deflection h ft. below the water surface, on the assumption that the horizontal arch alone carries the entire water load, is, in feet (See Fig. 38),

$$D_c = 0.434\ h\ k\ \frac{R_u\ R_{av.}}{b\ E} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (51)$$

where R_u and $R_{av.}$ are the up-stream radius and the mean radius of the dam at this elevation, respectively, and b is the thickness of the section. The coefficient k is given in Mr. Shirreffs' Fig. 22, reproduced herewith as Fig. 37.

Mr. Jorgensen, in a paper on "The Constant-Angle Arch Dam",‡ gives this formula as:

$$D_c = 0.434\ h\ k\ \frac{R_u^{\,3}}{b\ E\ R_{av.}}$$

but this is in error.§

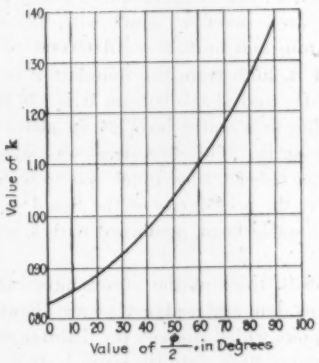


FIG. 37.

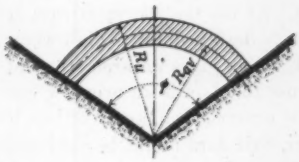


FIG. 38.

When the arch alone carries the entire water load, the arch compression is

$$c = 0.434\ h\ \frac{R_u}{b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (52)$$

* See Mr. McMillan's paper, already referred to.

† *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 166, Fig. 22.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 685.

§ See Mr. Shirreffs' deduction, *Transactions*, Am. Soc. C. E., Vol. LII (1904), p. 162, etc., and Mr. Woodard's remarks, p. 207.

in pounds per square inch, and the deflection

$$D_c = k\, c\, \frac{R_{av}}{E} \qquad (53)$$

in feet.

From Equation (51) it will be seen that with a given thickness of section, b, the crown deflection, D_c, varies practically as the square of the up-stream radius, and the stress, c, in Equation (52) varies as the up-stream radius. Or, keeping the stress constant, the deflection, D_c, decreases in the same proportion as the up-stream radius is decreased, and the width of the section also decreases in the same proportion.

Maintaining the extreme fiber stresses constant on the cantilever, the bending deflection increases directly as b decreases. Therefore, if R_u can be reduced to one-half, and the section also reduced to one-half, the permissible deflection, due to bending, will be doubled and the crown deflection of the arch will be half what it was before. The shear deflection would decrease a little, depending on how the load on the cantilever is distributed, but the advantage of decreasing the radius for the lower part of the dam is quite obvious.

If the section width is not reduced, the cantilever deflection is not changed by reducing the up-stream radius one-half, but the crown deflection of the arch would be reduced to one-fourth its former value. In other words, the constant-angle arch dam possesses the quality of giving practically the minimum arch deflection, besides that of giving the minimum volume, as pointed out by Mr. Jorgensen in the paper to which reference has been made.

In this connection, the Kerckhoff Dam, recently completed, designed by the writer and built by the San Joaquin Light and Power Corporation, Fresno, Cal., may be of interest. It is a constant-angle arch overflow dam* with fourteen Taintor gates, 14 ft. high by 20 ft. wide on top, and built to spill 60 000 cu. ft per sec. over the crest. It is approximately 94 ft. high from the foundation to the crest. At the top the up-stream radius is 205 ft., and at the bottom it is 136 ft.

It is designed for a high-water pressure due to a water level 24 ft. above the crest, which gives an arch stress of 225 lb. per sq. in. This low stress was selected because the dam is an overflow type, because ungraded tunnel muck was to be used for its construction, and, further, because, due to the peculiarity of the San Joaquin River, this dam might be subjected to the full static head, combined with a heavy overflow, while the concrete was still green.

At the base the Kerckhoff Dam is about 36 ft. thick at the center, tapering to 30.8 ft. at the abutments. A cross-section of this dam and a plan view are shown in Figs. 39 and 40. The reinforcing steel in the piers which support the Taintor gates is not shown. Some of this steel was carried about 20 ft. into the main body of the dam in order to prevent vertical tension acting on the concrete of the up-stream face, in case the top slab should be damaged, intentionally or unintentionally.

The constant-angle arch dam as built saved about 1 800 cu. yd. of concrete and 1 000 cu. yd. of river bottom excavation as compared with the best constant-radius dam which could be designed to fit the site and with the same arch stresses. An additional advantage is secured in that the shorter radii toward the bottom minimize the deflection near the bottom by compelling the horizontal arches to

* Built under a license arrangement with Mr. Jorgensen, the patentee.

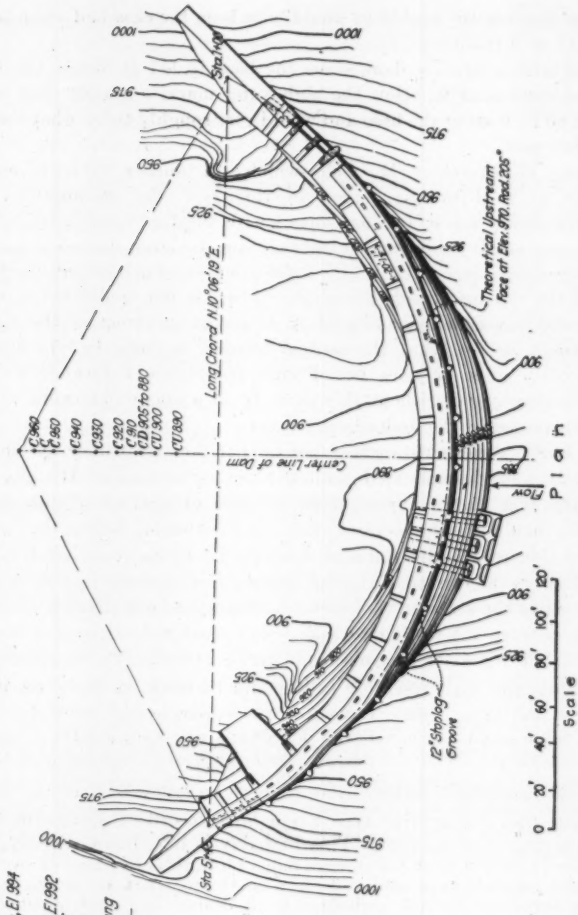

Long Chord N 10° 06' 19" E.
Center Line of Dam
Theoretical Upstream Face at Elev. 970. Rad. 205°
12" Stoplog Groove
Flow
Plan
Scale
0 20' 40' 60' 80' 100' 120'

FIG. 40.

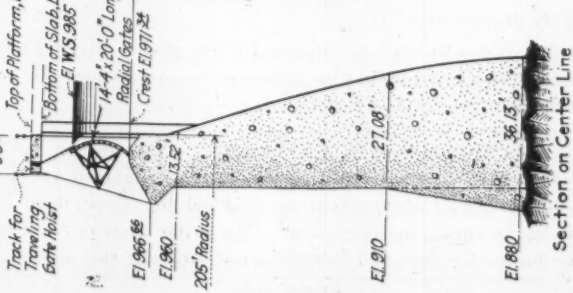

Top of Platform, El 994
Bottom of Slab, El 992
El W.S 985
14'-4" x 20'-0" Long Radial Gates
Crest El. 971.34
Track for Traveling Gate Hoist
El 966.66
El. 960
205' Radius
El. 910
El. 880
13.52'
27.08'
36.13'
Section on Center Line

FIG. 39.

carry a larger share of the load than would have been the case had a constant radius been used. (See Equation 53.)

Compared with a gravity dam, since the base is 117 ft. below the high-water mark and the crest is 24 ft. below the high-water mark, a gravity dam would have to be at least 80 ft. wide at the base and would cost roughly twice what the constant-angle arch dam cost.

The theory of Professor Moersch, which the author presents on page 26 is applicable to steel bridges, since in this case the assumptions made in developing the theory are in fair agreement with the facts, as the total foundation pressure is very small (compared with that due to large arched dams) and the modulus of elasticity is constant, at least for stresses such as are used in bridge design. For the reasons already given, this theory is not applicable to arched concrete dams, and any conclusions based on it are as incorrect as the assumptions.

It is difficult to see what the author intends to show by his discussion of "cylinder hoofs", since he does not discuss the elastic deformations which the dam would undergo, and without this there is no means of knowing what part of the load is sustained by "cylinder-hoof" action.

The one obvious and outstanding fact is that the stress distribution in curved dams is vastly more complicated than the author assumes. He has constantly overlooked the fact that a curved dam, whether of gravity section or having a smaller width than a gravity section, must fail in crushing before the dam can fail, and that for this reason the Rankine formula is the most satisfactory available, even though it is known that the resistance of the cantilever in bending will operate to modify the stresses, that is, to decrease the arch stresses.

It would be very interesting to know the actual deflection of a high straight gravity dam, but for such measurements to be of value the measurements with reservoir empty and with reservoir full should be made at the same temperature and nearly at the same time. Deflection measurements on arch dams can have only small value until the cantilever deflections are known with some degree of precision.

Regarding the author's statement on page 41, as follows:

"It is true that the gravity type has a long record of successful application, with comparatively few failures. This probably is due, however, only to the very fortunate circumstance, that for storage dams no high percentage of overload above the assumed load ever is possible. It is a matter of fact, however, that no other engineering structure of acknowledged good design has such a small factor of safety as a pure gravity dam."

It is odd that it should not have occurred to the author that the very fact that the ultimate load which can be put on a dam is known, accounts for the selection of a smaller factor of safety, and rightly so.

A. J. Wiley,* M. Am. Soc. C. E. (by letter).—The author has presented a most interesting and instructive discussion of the combined effect of vertical cantilever and horizontal arch action as modified by temperature changes and setting shrinkage in curved masonry dams. The writer wishes to express his appreciation of the unusually clear and forceful presentation of this subject, which is of

* Boise, Idaho.

special interest at this time when a number of arched concrete dams of greater height than ever before attempted are under consideration.

The author states his fundamental conceptions as follows:

"1.—That no curved dam of gravity section can be depended on to have any amount of water pressure supported by horizontal arching at the time of low temperature, unless the dam should have failed before in gravity action.

"2.—That probably all existing curved dams of reduced gravity section have failed in gravity action in that they have cracked either between the masonry and the foundation, or in the masonry itself.

"3.—That the usual method of calculating the stresses in arch dams by the well known 'cylinder formula' fails to furnish correct values."

He also states that the factor of safety of a straight gravity dam is much less than is generally assumed, and probably nearer 1 than 2.

In support of his first proposition the author calculates the deflection in an arch ring, due to the shortening of the arch by setting shrinkage, plus temperature contraction and compression from water load.

The writer does not think an allowance for setting shrinkage is usually necessary in concrete dams. He believes that the excessive shrinkage in the case cited by the author may have been due to the extremely rapid rate at which the concrete was placed, and possibly to the use of an excessive quantity of water in mixing. Where concrete is mixed very wet and placed at a rate giving a very rapid vertical increase of height, it might be expected that some abnormal action in setting would occur. The writer has seen liquid concrete poured from chutes at the rate of more than 1 ft. of vertical rise per hour in the forms. Such a rate of vertical rise combined with very fluid concrete should be avoided in dam construction. With an ordinary rate of vertical rise in placing and with a reasonable consistency in the concrete, the writer has never observed the effects of setting shrinkage in massive concrete, and thinks it is confined to thin sections which dry out in setting.

The subject of temperature changes in concrete dams of the gravity type was investigated in connection with the construction of the Arrowrock Dam and discussed in the paper by Charles H. Paul, M. Am. Soc. C. E., and the late A. B. Mayhew, Assoc. M. Am. Soc. C. E., on "Temperature Changes in Mass Concrete."* It is brought out in this paper that thermometers with electric recording devices were embedded in the concrete of the dam at distances from the faces varying from 1 to 76 ft., and the results up to January 1st, 1915, are shown by diagrams giving the temperature of the air and of the concrete at each of the thermometers. The readings of the thermometers were continued from that date to April 30th, 1917, and are now available.

An examination of these records shows that the temperature of the concrete near the face of the dam responded to changes in temperature of the air, but that the extent of the variation decreased rapidly as the depth from the face of the concrete increased. In detail, the effect of atmospheric temperature on the concrete at the various thermometers placed in the order of their depth from the surface may be summarized as follows:

Thermometer No. 9 was placed 1 ft. from the up-stream face. Except when submerged by the water in the reservoir, as was the case from May to August, 1915, inclusive, and from March, 1916, to the end of the record, it closely parallels a line

*Transactions, Am. Soc. C. E., Vol. LXXIX (1915), p. 1225.

which would be the average between the extreme high and low peaks in the temperature of the air, falling as much as 12° below the high points and failing to reach the low points in the mean daily air curve by as much as 20° when the low peak is of long duration and by 30° when the low peak continues only for a short time. The peaks in the concrete lag about a day, more or less, behind the air peaks.

Thermometer No. 7 was placed 2 ft. from the down-stream face and closely parallels the curve of Thermometer No. 9 from March 15th, 1915, to February 1st, 1916, with about 3° higher temperature. The temperature at Thermometer No. 7 for the winter months, like that of the more exposed Thermometer No. 9, parallels the extremes of temperature, keeping much closer to the maximum than to the minimum temperatures. The minimum temperature of the concrete is from 20° above the minimum to 30° above for short peaks, with a lag in time of about two to three days. The maximum temperature at Thermometer No. 7 is about 10° below the maximum mean daily air temperature, and the lag in time is about two days.

Thermometer No. 8 was placed 10 ft. from the down-stream face. It was affected by the setting temperature until about the end of 1915, after which it was entirely unaffected by monthly temperature changes and varied only with the season. Its lowest temperature in the winter of 1915-16 was 55° above the minimum mean daily air temperature with an apparent lag of about 45 days from the middle of the long cold period, which covered the month of January, to the beginning of the period of the lowest temperatures in the concrete on March 1st. The highest temperature at Thermometer No. 8 was about September 10th, 1916, and was 19° below the maximum mean daily air temperature on August 1st, which shows an apparent lag of 40 days.

Thermometer No. 10 was placed near the top of the dam 10 ft. from the up-stream face and practically the same distance from the down-stream face. It was probably affected by setting temperature through 1915. Its lowest reading for 1916 was 36°, which was 40° above the lowest mean daily air temperature. The lag in time between the middle of the cold period which covered the month of January and the low concrete reading was 22 days. The highest reading for Thermometer No. 10 was 19° below the maximum mean daily air reading, and the lag was about 45 days.

It is interesting to note that Thermometers Nos. 8 and 10, which are at the same distance from the down-stream face, have the same variation from the maximum air temperature, but that Thermometer No. 8 has a minimum reading of 15° higher than Thermometer No. 10 in the winter of 1916. Thermometer No. 8 also gave a reading of 12° above Thermometer No. 10 as the minimum for 1917. This difference between the action of the two thermometers may be accounted for by the fact that Thermometer No. 10 is separated from the air by 10 ft. of concrete on either side, while Thermometer No. 8 has this exposure on only one side and receives the internal heat of the dam on the other.

Thermometer No. 4 is 19.5 ft. from the main gallery of the dam in which the temperature is lower in summer and higher in winter than that of the open air. After losing its setting temperature, this thermometer kept a practically constant

temperature of 58° from June 1st to December 1st, 1915. It then dropped at a very uniform rate to 52.5° on August 1st, 1916, and maintained this reading to January 1st, 1917. On account of its being exposed to the gallery temperature instead of to the open air like the others, the results at this thermometer were not used.

Thermometer No. 3 is 20.0 ft. from the down-stream face. It lost its setting temperature about June 1st, 1915, and maintained a perfectly uniform reading of 55° through June, July, and half of August. By November 1st, it had risen to 57°, which it maintained until December 1st. This maximum of 57° is 23° below the season's maximum mean daily temperature of 80° and has a lag of about 45 days. From December 1st, 1915, to August 1st, 1916, Thermometer No. 3 dropped steadily to its minimum of 48.5° on the latter date. It seems probable that a part of this drop is due to the uniform fall in temperature of the entire body of the dam, which is shown by Thermometers Nos. 6 and 1 which are beyond the effect of any seasonal changes, but show a very uniform rate of fall. It would be more consistent to take the reading of Thermometer No. 3 on March 1st, 1916, as the minimum due to the seasonal variation. This is 50.5°, or 55° above the minimum mean daily air temperature. In the same way, the maximum for Thermometer No. 3 in 1916 may be taken as its reading of October 1st, which is 51°, or 31° below the maximum mean daily air temperature.

Thermometer No. 6 is 31 ft. from the up-stream and 61 ft. from the down-stream face. It attained its maximum setting temperature of 96° on October 1st, 1914, and dropped at a practically uniform rate, entirely unaffected by the air temperature, to 55° on May 1st, 1917.

Thermometer No. 1 is 76 ft. from the down-stream face. It reached its maximum setting temperature of 96.5° on October 1st, 1913, and dropped to 82° on January 1st, 1914. It then continued to drop at a uniform rate of 0.5° per month to 73° on June 1st, 1915, after which its readings were unreliable.

A summary of these results is shown in Table 16.

TABLE 16.

Thermometer No.	Distance to nearest face, in feet.	1915.				1916.			
		Lag, in days.	Temperature of concrete above minimum mean air temperature, in degrees.	Lag, in days.	Temperature of concrete below maximum mean air temperature, in degrees.	Lag, in days.	Temperature, of concrete above minimum mean air temperature, in degrees.	Lag, in days.	Temperature of concrete below maximum mean air temperature, in degrees.
9	1.0	1	20	..	*	..	30	1	*
7	2.0	2	30	..	†	2	35	2	10
8	10.0	..	‡	..	‡	45	55	40	19
10	10.0	..	‡	..	‡	22	40	45	19
3	20.0	..	‡	45	23	..	55	..	31
6	31.0	..	§	..	§	..	§	..	§
1	76.0	..	§	..	§	..	§	..	§

* Submerged.
† Not recorded because local conditions caused results not comparable.
‡ Affected by setting temperature.
§ Temperature of concrete not affected by air temperature.

These temperature changes cause contraction and tension in the dam when they are below the temperature of the concrete at the time it takes its permanent form, and cause expansion and compression when they are above this temperature. It is well known that concrete which is placed in winter is free from contraction cracks and that which is placed in summer almost invariably shows open cracks in winter. Therefore, it appears reasonable to assume that it is the temperature of the materials at the time of placing which governs the final form of concrete. Work on large masonry dams is usually carried on continuously except in the colder climates, where it is suspended only during the very coldest months. If it is assumed that the average temperature while concrete is being placed is 60°, contraction would occur only when the concrete is at a lower temperature. The range of temperature causing contraction at different depths in a concrete dam, on the foregoing assumption, is shown in Table 17.

TABLE 17.

Distance from face, in feet.	Temperature of Concrete, in Degrees.		Range of temperature, in degrees.
	Minimum.	Maximum.	
1.0	20	60	40
2.0	30	60	30
10.0	55	60	5
10.0	40	60	20
20.0	55	60	5
31.0	..	..	0

If these values are applied to a gravity concrete dam with the same section as the Arrowrock Dam, Table 18 shows the average temperature range causing contraction in horizontal bands or arch slices at various depths below the top, and in the last column is shown the average temperature range causing contraction in the whole body of the dam above the depth given in the first column.

Another consideration tending to reduce the temperature contraction is the difference in time lag at different depths. This varies, as shown in Table 16, from about 1 day near the surface to 45 days at a depth of 10 to 20 ft. At the time of minimum contraction near the faces of the concrete a temperature much above the minimum is prevailing in the interior, and by the time the interior has reached its minimum the concrete near the faces has reached a temperature much above the minimum. This lack of synchronism in temperatures and contractions at different parts of the dam has a most important modifying effect on contraction, and in view of this the figures given in Table 18 appear to be very conservative assumptions for the maximum temperature range causing contraction in the horizontal arch slices of a curved dam of gravity section at different depths below the top. From Table 18 it appears that the temperature range below an average "closing temperature" of 60° is 27° at the top, 20° at 50 ft., 11° at 100 ft., 5.5° at 200 ft., and 3° at 300 ft., below the top. From the last column it appears that, considered as a whole, a dam 80 ft. high would have a temperature range of 20° below the 60° "closing temperature", a dam 100 ft. high would have a temperature range of 17°, one 200 ft. high

TABLE 18.

Depth, in feet.	Thickness, in feet.	Average annual temperature range causing contraction, in degrees.	Mean annual temperature range causing contraction in whole dam, in degrees.
0	16	27	....
20	16	27	27
40	24	22	25
60	37	17	23
80	51	13	20
100	65	11	17
120	79	9	15
140	92	8	13
160	105	7	12
180	120	6	11
200	133	5.5	10
220	147	5	9
240	161	4.3	8
260	174	4	7.5
280	189	4	7
300	205	3	6.5

would have a 10° range, and one 300 ft. high a 6.5° range below the closing temperature.

There seems to be an inconsistency in the author's treatment of the division of the load between the arch and the cantilever. On page 13 of the paper, he says that the solution of the "problem of combined cantilever and arch action * * * consists in distributing by trial the water load on both cantilever and arches in such a manner that the deflections of identical points, calculated separately for cantilever and arch system under partial load and with due consideration of shrinkage and temperature deformations, are identical." This expresses the generally accepted assumptions regarding the division of the load between the arch and the gravity cantilever, but at the bottom of the same page he goes on to say:

"Case 1.—The cantilever is so strong that its deflection at the top, under the assumption that the cantilever takes all the load, is less than the deflection of the upper arch rings if these would have to sustain the entire water pressure (considering also the effect of shrinkage and temperature). In such a case (Fig. 4) practically all the load is supported by cantilever action, and horizontal arching may occur only when the arch is lengthened by a rise in temperature. Such conditions exist, for example, in a curved dam of gravity section."

It seems to the writer to be axiomatic that the only case in which the vertical cantilever action in a curved dam of gravity section would not be assisted in sustaining the water pressure by horizontal arching is that when the shortening of the arch by temperature or other contraction is so great that the entire load is carried by the gravity cantilever before its deflection brings any part of the horizontal arches into compression. Assuming a condition of elasticity in the concrete both vertically and horizontally, which implies, of course, that there is neither initial tension nor open cracks in the concrete, neither cantilever nor arch can sustain any part of the load without deflecting and in so doing transferring a part of its load to the other. If there are vertical cracks in the concrete, or if there is initial tension in the arch rings, the cracks must be closed or the tension relieved by the deflection of the cantilever before the arches can take any part of the load.

It is conceivable that a curved gravity dam might have such a flat arch or might be so long in proportion to its height that when the dam was contracted by cold

the whole load might be carried by the cantilever before the shortening of the arch by the deflection of the cantilever would close the vertical cracks or relieve the initial tension, but in the ordinary cases where the curved gravity dam is used, that is, for relatively high and comparatively short dams, a very considerable part of the water pressure is carried by arch action.

With the author's second proposition "that probably all existing curved dams of reduced gravity section have failed in gravity action in that they have cracked either between the masonry and the foundation, or in the masonry itself", the writer is inclined to agree to the extent of substituting "most" for "all". There may be some cases where the arch is so stiff relatively to the cantilever that the deflection of the cantilever is not in excess of that due to the release at its up-stream face of the compression due to its weight, but such cases must be rare.

The vertical cantilever is not a cantilever in the ordinary sense of being rigidly held at the base and resisting deflection by compression on one side of a neutral axis, with tension on the other. Its base is usually in a trench cut down to firm rock, but attached to it only by the adhesion of the mortar. It resists as a cantilever solely by the reduction in the initial compression, caused by its weight, at the up-stream face and its increase at the down-stream face. When the deflection of the cantilever is more than sufficient to relieve the initial compression at the up-stream face, tension results, together with a tendency to rupture its connection with the bed-rock or to open horizontal seams at the weakest points in the face. It is probable that the tension will be relieved by a separation from the bed-rock, as this is usually the weakest point, but this does not necessarily have a deleterious effect on an arch dam. There is probably an increased resistance to leakage because of the greatly increased unit pressure on the base due to the deflection of the cantilever which is now supported on a reduced base. The decreased resistance of the cantilever due to the vertical water pressure on the up-stream part of its base throws a greater load on the horizontal arches, but this is readily provided for if anticipated in the design.

The most serious effect of tension on the up-stream face is the increase of pressure at the down-stream toe. Arch dams are usually resorted to for very high dams where the unit pressures in a gravity dam would be reduced to the safe strength of the concrete by spreading the base. The pressures on the down-stream toe of a less-than-gravity type of arch dam, due to the reduced base width and caused by the deflection of the cantilever, should be a matter of grave concern. Although tension at the up-stream face is probably a common condition in thin arch dams of medium height and need cause no concern because the resulting pressures at the down-stream toe are well within safe limits, it is just as essential that a very high arch dam should be free from tension at its up-stream face as that a gravity dam should be so designed. The vertical cantilever should be tested for stresses in both faces, taking into consideration both arch and gravity action. It seems probable that when this condition is observed, very high arched dams will closely approach the dimensions of the gravity type.

The author's warning against the danger of closing vertical contraction joints by pressure grouting should be seriously considered, and this also applies to the expedient of preventing contraction by filling vertical sections of a dam at times of minimum contraction. Either of these methods will result in indeterminate and

possibly dangerous stresses in the vertical cantilever. The grouting process would be especially objectionable because it might result in very considerable compression in the dam at low temperatures increasing at high temperature with reservoir empty and resulting in an up-stream movement which might cause dangerous compressive stresses at the toe of the vertical cantilever at the up-stream face and tensile stresses at the down-stream toe.

The writer agrees with the author's statement that the ordinary "cylinder formula" fails to furnish correct values of all the stresses in arched dams. It does give, however, approximately correct values of the stresses in the arch ring when carrying that part of the total load which it receives from the division of the load between the arch and cantilever. The apparent paradox of the maximum deflection at the top where the cylinder formula indicates there should be no stress is, of course, due to the transference of stress from the lower arch rings.

In this connection the writer thinks there must be an oversight in the author's statement on page 11 "that in the common type of arched dam, for which the radius of the center line of the horizontal arch slices is more or less the same, the deflections, D at the crowns are approximately the same for the lowest arches as for the highest ones, irrespective of the length, provided the unit stresses in all arches are the same."

The author's statement that the factor of safety in a straight gravity dam is probably nearer 1 than 2 is rather startling. One reason given for the statement is that vertical arching takes a part of the weight from the highest part of the dam to the sides of the canyon, and that a similar effect is produced by inclined cantilever action. It appears to the writer that a transference of stress from the highly stressed toe of the dam in the deepest part of the canyon to the less highly stressed sections would tend to increase rather than diminish the factor of safety. The tendency to concentration of stresses at the sides of the canyon would be prevented by a continuance of the diagonal cantilever action from which there would result a progressive transference of stresses from all the higher to the lower parts of the dam, with a resulting decrease in stresses in the most highly stressed part of the dam. Such vertical arching and diagonal cantilever action, however, would be prevented to a great extent by the contraction joints with which all high concrete dams are now provided.

The principal argument of the author to prove the low factor of safety in gravity dams is that horizontal cracks or seams may be opened in the up-stream face or beneath the base by an increase of 30 to 50% in the head for which a gravity dam has been designed. Such a condition the writer believes to be beyond the realms of possibility. The stability of all gravity dams is calculated with a water level well above any possible flood and to question their safety with such an impossible increase of head is no more reasonable than it would be to question the safety of any other structure under a load which could not be applied.

To overcome the question raised by the author against the safety of straight gravity dams or curved dams of reduced gravity section he proposes a new type, the "cylinder-hoof", consisting of a single arched dam of gravity section, or a series of arch dams, which derive their stability from being required to act as a whole.

Without going into the merits of this new type it would seem to be difficult to prevent it from being broken into sections by vertical contraction cracks if not provided with contraction joints, either of which conditions would defeat its purpose.

It appears to the writer that in designing any arched dam of either full gravity, or reduced gravity, section all stresses should be considered as transmitted in either vertical or horizontal planes. Nearly all high dams are now constructed of concrete and they are of necessity provided with construction joints if not contraction joints. If they are not so provided, they are sure to crack into sections at points which cannot be determined in advance. It is impossible to build, or at least to maintain, a large dam as a monolith. In addition to the necessary vertical contraction or construction joints, there are numerous horizontal joints between successive days' work and many others where work has been suspended on certain sections of the dam for a few days or weeks. All these joints form planes of weakness, as can often be seen by the seepage along such horizontal planes when a thin concrete dam is first subjected to water pressure, or by the ease with which concrete breaks along horizontal planes when a wall is being demolished.

The only directions in which stresses in a curved concrete dam can be transmitted with certainty are in vertical planes normal to the faces or in horizontal planes. There is no conceivable condition under which the properly proportioned vertical gravity cantilever and the horizontal arch, acting either together or singly, cannot transmit any stresses they may receive, and although under certain conditions, stresses may be transmitted in other directions, as through diagonal cantilever or vertical or diagonal arch action, such action is altogether uncertain, and serves only to relieve the calculated stresses in the vertical cantilever and horizontal arch.

CHAUNCY WERNECKE,* ASSOC. M. AM. SOC. C. E. (by letter).—The whole subject of the theories of masonry dam design is again laid open by the author's paper. It is very commendable to have these theories reviewed and revised occasionally, so that progress can be made toward a complete understanding of the stress action in dams.

The paper as a whole has much of real value, and the mathematics have been reduced to a practical point. The manner of curtailing the mathematics by using approximations within the usual limits of practice, the introduction of approximate but simpler extraneous relations, and the application of the well known graphical method for easily determining the deflection of beams with variable moments of inertia, make the paper readable and useful to many engineers.

It can be shown on the basis of tests on boiler tubes that the "cylinder formula" fails to give correct values for stresses. Mansfield Merriman, M. Am. Soc. C. E., gives† the results of three tests on boiler tubes, which independently verify an empirical formula developed by Wood. The formula is:

$$P = \frac{9\,600\,000\ t^{2.18}}{l\,d}$$

where P = the unit pressure, t, the thickness, P, the length, and d, the diameter, all in inches. The diagram (Fig. 41), is constructed on the data of the largest

* Seattle, Wash.

† "Mechanics of Materials."

test noted by Merriman; a wrought-iron tube 60 in. long, 18¾ in. in diameter, and of a thickness of ¼ in., collapsed under an external pressure of 420 lb. per sq. in. The formula gives 409 lb. per sq. in. Using 420 lb. in the formula, $t = \frac{R\,P}{q}$, the value of q is 15 750 lb. per sq. in. Using these values and varying the radius, the straight line of the diagram results.

Hence, the comparison between the cylinder theory formula and the actual tests shows a wide variation from correct values. This is to be expected when it is considered that, due to inequalities of material and workmanship, the tube cannot hold its absolute circular section, but soon becomes an ellipse when under pressure. This ellipticity, however slight, causes bending in the plane of the major axis and hence the moment of inertia of the longitudinal section becomes a factor. As the moment of inertia varies as the cube of the thickness, it is evident that any increase in thickness very rapidly increases the strength or allows a rapid increase

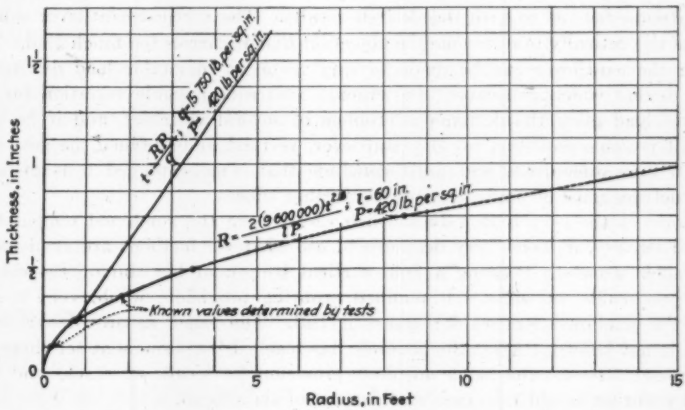


FIG. 41.

in the radius for the same load. Since the cylinder formula fails to give correct results for the stresses in a boiler tube—a structure which is certainly a cylinder—it would show a greater variation when applied to an arch dam which, at best, can be considered only a small irregular part of a cylinder considerably restrained on three sides, and, further, seriously affected by gravity action. However, Fig. 41 shows that the formula errs considerably on the safe side, when it is assumed that the action is entirely arch action.

The assumption that the bed-rock is unyielding seems to be extreme. In the calculations on the Zola Dam, the author uses $E = 3\,500\,000$ lb. per sq. in., because of the very good quality of rubble masonry. A degree of elasticity is thus recognized for natural rock, and it seems only logical that this could be applied to the rock in its native position as well. In this respect, the calculations on the Lake Spaulding Dam should have included a consideration for the concrete gravity base where, no doubt, there is an appreciable yielding. However, this point will probably not alter the author's conclusion that many existing arch dams have a cracked

cantilever, as a yielding bed-rock applies to the arch abutments as well as to the base of the cantilever.

The writer, following the method given, has checked a high arch dam of recent design, and has found a stress in the toe approximating 50 tons per sq. ft. and a tension on the up-stream side of nearly 200 lb. per sq. in. Does this mean that the dam has cracked? Such concrete will stand more than 200 lb. per sq. in. tension due to bending.* However, this does not argue for the safety of the dam, but rather the opposite, since a tensile stress of 200 lb. per sq. in. is so near the rupture point that, should rupture occur, the sudden distribution of the load and shift to arch action might prove disastrous. This could be overcome by destroying the cantilever action tensile stresses by constructing the arch in horizontal sections with fairly smooth contact surfaces. This would insure maximum arch action and reduce cantilever action to that giving only compressive stresses. It would be better to destroy only partly or reduce the full cantilever action by localizing the tension cracks at horizontal construction joints extending such a distance from the up-stream face as to give the desired results. The vertical cantilever can be cut so as theoretically to overcome the objection that it carries too much load. The load for the cantilever can be made to vary from its inevitable load due to its weight, to that which will cause it to crack. This wide possible variation for the cantilever load gives the designer a problem of several variables, and if he considers all possible positions for the cantilever, vertical and inclined, he gets into rather serious difficulties, and must conclude that some extended tests on arch dam structures must be made to guide his further effort.

Because of the very high values which result from the combined temperature and load stresses, it seems very desirable to use whatever methods are available to reduce such stresses. Pouring a final vertical section or key during the coldest season practicable, or some well planned grouting provision, would very largely reduce the combined stresses for reservoir full. The exact nature of shrinkage stresses is not known. After the concrete has reached its permanent set there are no shrinkage stresses, and hence adequate provision for localizing cracks and subsequent grouting would take care of this part of the design.

The writer concurs most heartily with the author's plea for stress, temperature, and deflection measurements on arch dams. His suggestion that the State authorities require adequate provisions for such measurements is a good one, and ought to be easy of application.

EDWARD WEGMANN,† M. AM. SOC. C. E. (by letter).—The author deserves much credit for the lucid manner in which he has investigated the stresses existing in curved masonry dams, and for developing formulas and graphical methods for the approximate determination of these stresses. As stated in various parts of the paper, these stresses, which result from arch action, cantilever action, and, near the base of the dam, from wedge action, are exceedingly complex.

Most of the formulas developed by the author are based on the assumption that the dam is composed of homogeneous masonry, having a known value of E, the modulus of elasticity. For a dam built of concrete made with uniform, small-size aggregate, these formulas may be sufficiently exact. In ordinary practice, however,

* "Concrete Plain and Reinforced", by Messrs. Taylor and Thompson, Third edition, p. 334.

† New York City.

stones varying in size from 1 cu. ft. to 3 cu. yd. are placed in the concrete. In cyclopean masonry, which is frequently used, the large stones form from 20 to 40% of the masonry. Such masonry cannot be considered to be a homogeneous monolith and certainly has no definite value of E. In many dams, the laying of the masonry consumes several years, part being laid in summer and part in cold weather. The effects of temperature and shrinkage may vary, therefore, very considerably in different parts of the dam. For these reasons the writer doubts whether it is possible, in the present state of knowledge, to find mathematical formulas which will determine closely the stresses in a curved masonry dam, built in the usual manner of concrete and large stones. Even if the dam was built of rubble, it is doubtful whether a close value for E could be assumed. After careful observations of the behavior of curved dams, subjected to varying water pressures and changes of temperatures, have been made, formulas may be devised which will apply to stresses in these dams, or similar ones, but great uncertainty will remain in the use of these formulas for dams where conditions may be different.

Most engineering formulas are based on some simple hypotheses, which are only approximately correct. If structures designed by these formulas are found to be safe, engineers are justified in concluding that the assumptions made were sufficiently close to the actual conditions for practical purposes.

The ordinary formulas used in designing the profile of a gravity dam have been devised in this manner. They are based on the assumption that the stresses in a dam, reservoir full or empty, are distributed as a uniform, or a uniformly varying, stress. This distribution, which is only approximately correct, exaggerates, in all probability, the pressures in the masonry near the faces of the dam, and underestimates them for the central part. In this respect the formulas err in the direction of safety. No dam designed according to this theory has been ruptured on account of weakness of profile. The comparatively few failures of masonry dams which have occurred were due to faulty construction or to upward pressure under the base of the dam, for which proper allowance had not been made.

Thus far, most curved dams have been designed by using the simple cylinder formulas, $T = p\,r$, in which T = uniform thrust in a circular ring, p = pressure per unit length of the ring, and r = radius of the ring's outer surface. As pointed out by the author, this formula is certainly not theoretically correct, and yet, among the many bold designs of curved dams made by applying this formula, not one has failed. The Zola Dam, built in France in 1843, was, as far as the writer knows, the first curved dam built in accordance with this formula. The famous Bear Valley Dam, in California, was one of the boldest designs of curved dams. It had a top width of 3.17 ft., and at a depth of 48 ft. below the crest, the dam was only 8.42 ft. thick for a normal pressure of 44 ft. of water. This dam was replaced in 1910-11 by a multiple-arch concrete dam, built about 200 ft. farther down stream.

In New South Wales,* thirteen curved concrete dams were built in 1896-1906. They were all designed by the cylinder formula, and have bold profiles. The pressures in the masonry in these dams vary from 10 to 25 tons per sq. ft. The highest, known as Lithgow Dam No. 2, has a maximum height of 87 ft. above the foundation. It is 221 ft. long on the crest, 3 ft. wide on top, 24 ft. wide at the base, and is curved to a radius of 100 ft.

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXVIII, p. 1.

In order to insure sufficient strength against sliding or shearing, and to make a water-tight dam, the profiles could not be made much thinner than those of the dams mentioned. In course of time a better distribution of the masonry, however, may be found by observations on dams, and as a result of investigations like those made by the author. The effect of changes in temperature on the stresses in the masonry of a curved dam was studied by the late George Y. Wisner, M. Am. Soc. C. E., and Edgar T. Wheeler, M. Am. Soc. C. E., in the design of the Pathfinder Dam, as mentioned by the author.

In conclusion, the writer would say that although he admires the mathematical skill shown by the author and considers the paper a step forward in the right direction, he does not think that the practical solution of the complex problem of the proper design of curved dams can be made until engineers have more data on the behavior of such dams and on the values of the modulus of elasticity of the masonry.

D. C. HENNY,* M. AM. SOC. C. E. (by letter).—The author's paper is interesting as a mathematical discussion of hypothetical parts of a complicated problem and follows much the same lines of inquiry as previously published studies.

The part of the paper with which the writer finds himself in closest accord occurs on page 45 under the head of "Curved Dams as Cylinder Hoofs", and is quoted below:

"* * * The internal stresses in a cylinder-hoof dam, due to the water pressure and to the temperature deformations, etc., are of a very complicated nature. In addition to a combination of vertical cantilevers and horizontal arches, there exists in dams of considerable thickness a wedge action in the lower portions where the width of the canyon is small as compared with the thickness of the dam; so that under no circumstances can this part be considered as arching. In the middle and upper portions of the dam where horizontal arch action may occur, the deflection of the cantilever produces torsion in the arch rings, and any horizontal dam section, after the bending of the cantilevers, is warped after deformation. This bending is a maximum in the center of the dam and zero at the side-hills, and these torsion stresses, which hardly can be determined mathematically, tend to stiffen the cantilever and to decrease the deflections of both the cantilever and the arches.

"Also, the maximum compressive stresses of the entire structure have a tendency to follow the lines of greatest resistance, which in arch dams in general are not horizontal even in the purely arching parts, but are somewhat inclined, with the tendency to form arches of minimum radius and span, between any point in the middle of the dam and the nearest points of the unyielding side abutments.

"These torsion and inclined arch stresses, in general, are of too complicated a nature to be determined; also, they are comparatively only small. Both tend to stiffen a curved dam, and may be considered as increasing the safety of the structure. In a practical design, therefore, these complicated stresses may, in general, safely be neglected, and the dam investigated only with regard to combined vertical-cantilever and horizontal-arch action. As already mentioned, the results will be the more accurate the thinner the dam compared to its height."

The foregoing quotation presents the problem in a manner admirable both for its comprehensiveness and its conciseness. The only part to which the writer takes exception is the next to the last sentence. It is there inferred that the mathematical requirements of a satisfactory solution are most nearly met by considering combined cantilever and arch action in the manner proposed by the author.

* Portland, Ore.

It was elsewhere stated in the paper (page 24) that applying the author's method to the well-known cases of the Pathfinder and the Shoshone Dams it is found that excessive tension in the masonry due to cantilever action must have produced cracks near the base. The writer has had some connection with the design of these dams, and is in no way convinced that such cracks exist or that the design is fundamentally wrong or in any way dangerous.

The reason that the mathematical treatment by the author carries no conviction is that, owing to the complexity of the problem, it is extremely incomplete and ignores numerous important elements to which he himself, in the part of the paper here quoted, makes able reference.

In a lighter-than-gravity arch dam the simplest case is that of a dam of uniform height with vertical, radially inclined abutments. The support given to the arch by cantilever action is maximum in the center and reduces toward the abutments. On the other hand, the support given to the arch by shear or friction along radially vertical planes is maximum at the abutments and vanishes toward the center. It follows that the load remaining to be carried by the arch has a certain value at the center, a greater value between the center and the abutments, and again a lesser value close to the abutments. In other words, even in such a simple case the arch, if cantilever and shear effects are allowed for, is unevenly loaded, a result which has nowhere been carried into the author's formulas.

The effect on the central part of the dam of support from the sides, through shear or friction, becomes more marked in the usual case of an arched dam in a V-shaped canyon, which for lighter-than-gravity section dams is the more usual case. For such dams no reasonably accurate refinements can be expected in the results of a mathematical analysis if no cognizance is taken of the following factors:

(*a*)—Arch or beam effect in transferring weight downward, which may completely upset vertical cantilever load calculations.

(*b*)—Shear or frictional support from the sides of the canyon for the assumed vertical cantilever slices.

(*c*)—Deviation from horizontal arch action.

(*d*)—Shear or friction between assumed arch slices.

The foregoing argument implies that, especially as regards dams in V-shaped canyons, the section of the canyon must become an integral part of any satisfactory mathematical determination of stresses. It is for this reason that, for instance, the author's positive conclusion that a crack must exist on the water side near the base of the Pathfinder Dam seems unwarranted. The direction which such crack might be expected to take is, moreover, difficult to conceive, as there is no great extent to any horizontal plane in that part of the dam. It may also be well to state that no evidence of such crack exists, so far as the writer is aware.

The partial solution of this problem offered by the author can therefore hardly be considered by the engineer charged with the responsibility of dam design as a dependable or satisfactory guide. Past practice in the design of arch dams of lighter-than-gravity section is believed to have been based fundamentally on the use of the simple arch formula. Elaborate studies have been made of possible

secondary stresses, but these were in a sense *ex parte,* and so far as the writer knows did not control the general design.

The practice just mentioned has been continued with full knowledge of the shortcomings of the arch formula as not representing correctly, either the direction or the intensity of actual stresses. There has been, however, the underlying conviction, as expressed by the author, that certain secondary stresses may "tend to stiffen a curved dam and may be considered as increasing the safety of the structure", and that should such stresses be locally excessive and cause rupture, the planes of breakage are likely to be either horizontal or else radial, neither of which would interfere with the arch action upon which the design is based.

The excessive stresses which may result from temperature changes frequently have been the subject of study. The adoption of construction methods to control them is believed to be of greater practical importance than any refinements of calculation over the simple arch formula which so far have been offered. The foregoing statement is made with full appreciation of every thorough effort to analyze this complicated problem and reduce it to comprehensive mathematical formulas which would permit with confidence the use of stress limits more nearly approaching those in every-day use in reinforced concrete construction.

WILLIAM A. MILLER,* ASSOC. M. AM. SOC. C. E. (by letter).—The author's Equation (15*a*) gives the temperature thrust, with regard to H and V of Fig. 7*b*, if flexural deformations only are considered. If the effect of direct stress is included,

$$H_t = \pm \frac{E\,c\,T\,l}{\int_{-\frac{l}{2}}^{+\frac{l}{2}} \frac{y^2\,d\,L}{I} + \int_{-\frac{l}{2}}^{+\frac{l}{2}} \frac{\cos^2\phi\,.\,d\,L}{A}}$$

where A = the cross-sectional area of the rib and ϕ = the angle which the rib axis makes with the XX axis (Fig. 7*b*).

The second term in the denominator is unimportant in a thin rib of high rise, but for the thick, flat rib which is common in the arch dam, its omission brings about a considerable over-estimate of temperature thrust and moment.

Applying the author's approximation for flat arches, that $d\,L = d\,x$,

$$H_t = \pm \frac{E\,c\,T\,l}{\frac{1}{I}\int_{-\frac{l}{2}}^{+\frac{l}{2}} y^2\,d\,x + \frac{1}{A}\int_{-\frac{l}{2}}^{+\frac{l}{2}} \cos^2\phi\,.\,d\,x}$$

For a rise of 0.10 of the span, the writer, in analyzing a series of arch ribs, has found that $\int_{-\frac{l}{2}}^{+\frac{l}{2}} \cos^2\phi\,.\,d\,x$ may be taken as $0.95\,l$. The equation for temperature thrust may then be written:

$$H_t = \pm \frac{E\,c\,T\,l}{\frac{12}{t^3}\cdot\frac{4\,h^2\,l}{45} + \frac{0.95\,l}{t}} = \pm \frac{E\,c\,T}{1.07\,h^2 + 0.95\,t^2}\cdot t^3$$

* Sydney, New South Wales, Australia.

The ratio of H_t found by the author's equation to the value found by that of the writer is:

$$\frac{1.07\,h^2 + 0.95\,t^2}{1.07\,h^2} = 1 + 0.89\left(\frac{t}{h}\right)^2,$$

or, the percentage over-estimate of temperature thrust, using the author's equation, is $89\left(\frac{t}{h}\right)^2$. For a case with a span of 400 ft., a rise of 40 ft., and a thickness of 40 ft., the thrust is 89% too great. For a rise of 0.15 of the span, $\int_{-\frac{l}{2}}^{+\frac{l}{2}} \cos^2\phi \,.\, d\,x$ is approximately 0.90 l, and the percentage over-estimate of thrust is $84\left(\frac{t}{h}\right)^2$.

L. Luiggi,* M. Am. Soc. C. E. (by letter).—On the question of gravity and arch action in curved dams, A. F. Parker, M. Am. Soc. C. E., among many interesting remarks, states (page 68), that "the Crowley Creek Dam * * * designed with a radius of 70 ft." and with a thickness on top of 3.6 ft., constructed of "concrete far from good", and with the "alignment so seriously at fault that the radius at the top varied from a minimum of 58 ft. to a maximum of 80 ft.", and with a "crack ⅜ in. wide at the top when the dam was empty in cool weather", * * * "nevertheless, floods to a depth of 2 ft. had passed over the top of the dam, carrying large blocks of thick ice and still the dam did not fail." * * * "This example cannot fail to cause wonderment as to what load of water pressure a well built, true, and symmetrical thin arch really could safely carry."

To this query the writer can answer by mentioning the case of an extremely thin arch which stood successfully a head of water of 4 m. (13.1 ft.).

In the multiple-arch dam on the River Scoltenne, in North Italy†, an opening 3 m. (nearly 10 ft.) wide was left in one of the arches near the foundation, in order to allow the water to pass during construction. This opening was afterward walled up and coated on its up-stream face with cement mortar, 3 cm. thick (1¼ in.), in a manner similar to the remainder of the up-stream face of the dam. After some months this hole had to be opened again. The opening was made very carefully, as there was still over it a stratum of water 4 m. (13.1 ft.) deep. The masonry was chiseled away, leaving the plastering untouched. This plastering withstood alone, perfectly, the pressure of the water above. When a round hole was pierced through this thin arch of plastering with a pick, a round jet of water poured out violently, and the thin arch continued to stand firmly the pressure of the water above.

It would be worth while to repeat such an experiment and to calculate the stress caused in the mortar forming this thin arch. If the usual simple formula, $s\,t = H\,d\,R$, is applied—in which, s is the stress, t the thickness of the arch, H the head of water, d its density, and R the radius of the arch—it is seen that a stress of about 56 kg. per. sq. cm. (795 lb. per sq. in.) at least was produced in the thin arch.

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* Rome, Italy.

† For a complete description of this very interesting multiple-arch dam—the first built in Italy—see *Elettrotecnica*, Milan, Vol. VII, No. 27, September 25th, 1920.

This example, perhaps, is the extreme limit to which arch action can manifest itself, and, indeed, it causes "wonderment as to what load of water pressure a well built, true, and symmetrical thin arch really could safely carry," as Mr. Parker justly remarks.

Fig. 42 in a sketch of the conditions which existed at the Scoltenne Dam, which is about 70 ft. high.

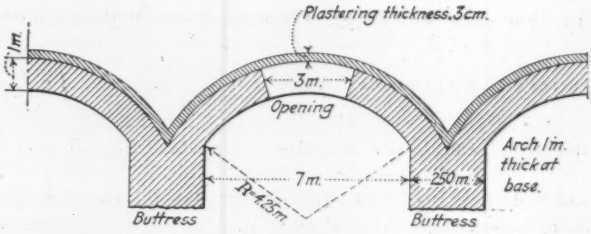

FIG. 42.

In view of the excellent results given by this dam, another similar one, but with a height of 200 ft. above the foundations, is in course of construction on the River Tirso, in Sardinia, Italy, which, perhaps, will be the highest multiple-arch dam yet attempted anywhere.

HUGO RITTER,* ESQ. (by letter).—In a scientific determination of the stresses in an arch dam, the influence of the weight of the structure (gravity or cantilever action) naturally should be duly considered in addition to direct horizontal arching. The author's assumption of combined cantilever and arch action has been recognized previously by others as being sound in principle in its application to arch dams. The fact that the method permits investigation not only of a section at the crown of the arch, but equally well any other section closer to the arch abutments is an advantage which will be appreciated, particularly in the design of arch dams with a long crest and in canyons of irregular cross-section. As in the design of other structures which are statically indeterminate, the division of the load between assumed elementary systems so as to effect the coincidence between the deflection lines of the assumed elementary systems, proves to be the easiest solution also in the design of arch dams.

The author's graphical method is based on trial computations. These calculations may be shortened and the exact coincidence between the deflection lines of cantilevers and arches can be obtained directly in a single operation. This method is based on the following fundamental principle:

"The elastic line of a beam, which corresponds to the load, $P = 1$, acting in any point on this beam, is the line of influence for the movement of this point for the unit load."

This principle, applied to the problem of arch dams, works as follows:

Divide the arch dam by horizontal sections into a number of arch slices, say, n. Draw all the elastic lines of the cantilever which result from the application of the unit load, $P = 1$, in the center of the n parts of the cantilever, using the graphical method shown by the author. In this way n influence lines are obtained.

* Zurich, Switzerland.

Compute now the deflections of the n arch rings by applying the unit load, $P = 1$, to the arches. Multiply the ordinates which correspond to the n parts of the cantilever, for each one of the n lines of influence, with the corresponding unknown partial cantilever loads, whereby the deflections of the n points of the cantilever, in which the unit load was acting, are obtained. These deflections must equal the arch deflections for the remaining load, and this condition leads to n equations with n unknown quantities. The solution of these equations furnishes the cantilever loads. The load acting on the arches is obtained by deducting the cantilever load from the total water pressure.

The stresses in both cantilever and arches may then be calculated in the usual way, whereby for the cantilever, the vertical loads (weight, vertical component of water pressure, etc.) also have to be considered.

In a similar manner, the investigation may be extended to any vertical section through the dam nearer the side-hills. It will be found that the distribution of the water pressure is often uneven over the same arch slice. This is particularly evident in the case of canyons with side-hills very nearly vertical, and also for canyons of very unsymmetrical cross-section. Such unsymmetrical loading of the arches, however, is of little importance, as calculations will show, and this approximate method of calculating the stresses in arch dams is accurate enough for practical purposes.

The influence of temperature and shrinkage deformations on the division of the water pressure between cantilevers and arches is preferably taken care of by considering such deformations in the same manner as if they were produced by additional water pressure. Such an imaginary load may again be represented as a load area, part of which would act on the cantilever and part on the arches.

It is advisable to figure the loads and stresses for arch dams as accurately as possible, and to take care of all tension stresses which may occur by a sufficient quantity of steel reinforcement.

FRED A. NOETZLI,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer was delighted to read Mr. Cain's discussion and to follow his ingenious method of comparing the results of Mr. B. A. Smith's analytical theory† with the writer's graphical trial method. The remarkably close coincidence of the final results obtained by each method, as evidenced particularly by the figures of Tables 7, 9, and 10, ought also to convince the most skeptical that both theories are fundamentally correct. The fact that the calculated deflections check very closely with actual measurements made for the Wooling Dam also shows clearly the soundness of the assumptions underlying those theories. Thus, either Mr. Smith's analytical or the writer's graphical method of arch dam design may safely be used in the future.

Mr. Cain's detailed example of a practical design and his method of decreasing the number of trials by interpolation, as illustrated by Fig. 17, will be appreciated by many practical designers. However, Dr. Ritter's method of obtaining the complete coincidence between cantilever and arch deflection lines in one operation may often be used to advantage.

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* San Francisco, Cal.

† "Arched Dams", *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

In connection with Mr. Cain's suggestion to tabulate the coefficients of Equations (13), (13*a*), (13*b*), etc., for varying values of $\frac{L^2}{h}$ in order to get more accurate results, the writer is of the opinion that the advantage gained thereby would probably be outweighed by the fact that the formulas would lose somewhat of their simplicity, which appeals so much to the practical designer. At the present time, when the fundamentals of his theory are still criticized and attacked, he thinks it advisable to keep all equations as simple as possible even if it means sacrificing a small percentage of their accuracy.

The same reasoning led the writer also to propose his assumption of temperature variations, with which Mr. Cain is not able to agree. In fact, Mr. Cain's objections to the assumption that in most dams the variations of temperature may be taken as a maximum at the crest and diminishing uniformly to nothing at the base, are well founded. For low, thin dams the seasonal variation of temperature is probably about the same from the base to the crest. In high dams of considerable thickness, however, the variation in temperature is much less at the base than near the crest. The temperature measurements made for the Arrow Rock Dam, as cited by Mr. Wiley, showed a mean annual temperature range of 27° Fahr. at the crest and only 6.5° Fahr. at the base of this dam. Only extensive temperature measurements on slender arch dams can lead to definite assumptions. For the sake of simplicity, the writer assumed a uniformly decreasing variation from the crest to the base.

With regard to a dam not "fixed" at the base, the writer is unable to agree with Mr. Cain (page 87) that "the conditions to be fulfilled are: The moment and shear at the crest shall equal zero, and the moment and displacement at the base shall likewise equal zero."

There will nearly always exist a certain moment at the base of a dam due to the weight of the masonry above. This moment may be smaller than the moment due to the water pressure, so that such a dam is lifted away from the foundation on the up-stream side. If water enters into the open crack a further moment of opposite sign to that due to the weight of the masonry is produced by uplift. The difference between these two moments still restrains somewhat the lower part of the dam. For this reason, Mr. Smith's method for this case is only approximate. If the resulting moment could be determined accurately, both Mr. Smith's and the writer's methods would be applicable, that is, the writer's method by trial until the angle of the tangent to the deflection curve at the base has been determined so that both cantilever and arch deflection lines coincide again for the entire height of the dam. The cantilever is thereby not absolutely fixed, but only partly restrained by the predetermined moment at the base.

Certain remarks of criticism made by several members with regard to "Curved Dams as Cylinder Hoofs" were no doubt caused by lack of clearness in the writer's presentation of this subject. Cylinder-hoof action refers only to the external stability of a curved dam (soil pressure resistance to overturning). The internal stresses in the dam body are supposed to be investigated separately by means of the method of combined cantilever and arch action.

In a true arch dam founded on solid rock the external stability is apparently practically always established, as there is hardly any danger that excessive founda-

tion pressures will be produced near the dam wings due to a tendency of a dam to overturn.

In a curved dam of full gravity section, however, the conditions are just opposite, except for dams in narrow canyons. The internal stability for gravity dams is apparently fairly well established, as the cantilever action alone is supposed to be sufficient to carry all the water pressure. If the cantilever action should be partly destroyed, say due to a defective foundation near the downstream side at the center of the dam, or due to uplift, etc., arch action would undoubtedly tend to prevent a failure. The question then is whether a real failure would be more likely to occur by crushing the arch or by producing excessive pressure near the dam wings. Suppose, for the sake of argument, such a dam to be doomed to failure. In narrow canyons with side-hills of solid rock, the masonry near the center of the dam probably would be crushed before the masonry or the rock near the dam wing would give way. A straight gravity dam at the same location most probably would fail near the center by shearing out a longer or shorter vertical dam slice near the place where, say, the defect in the foundation had occurred or where uplift had weakened the stability of the structure. Engineering instinct has taught long ago that most probably the straight gravity dam would fail before a curved gravity dam under similar circumstances. For this reason most engineers build curved gravity dams in preference to straight ones, and rightly, because they believe that the factor of safety of a curved gravity dam is higher than that of a straight one. It is one of the purposes of the writer's theory of "cylinder-hoof action" to provide means to determine mathematically, at least in an approximate way, how much larger the factor of safety of a curved gravity dam is as compared to the factor of safety of a straight dam of the same cross-section at the same location.

Cylinder-hoof action is further of particular importance for dams across valleys the width of which is, say, over ten times the height of the dam, and where the long span would seem to necessitate a straight dam. In such a case, undoubtedly, a series of arches of comparatively short spans of, say, four to five times the height of the dam are much to be preferred to a straight gravity dam, as an investigation, based on the principles of cylinder-hoof action, will show.

The writer's statement that the factor of safety of a long straight dam of standard gravity section is probably nearer 1 than 2 has received various comments from different members. Mr. Jakobsen thinks "* * * that the very fact that the ultimate load which can be put on a dam is known, accounts for the selection of a smaller factor of safety, and rightly so." On the other hand, Mr. Wiley believes that a load increase of 30 to 50% is "* * * beyond the realms of possibility * * * and to question their safety [gravity dams] with such an impossible increase of head is no more reasonable than it would be to question the safety of any other structure under a load which could not be applied." The same reasoning, of course, applies to arch dams, but who would advocate unit stresses of 1 200 to 1 500 lb. per sq. in. for such structures?

On the other hand, it is a fact that some gravity dams have failed. The reason for this may be due less to weakness of the profile than to faulty construction or to upward pressure under the base of the dam, as pointed out by Mr. Wegmann. Other failures are known to have occurred

from undermining of the toe of the dam by water flowing over the crest or through spillways in the side-hills. One of the highest gravity dams in the country was undermined by the whirlpool of the waters spilled through the spillways on both sides of the dam. A hole 60 ft. deep was formed in a gravel pocket at the toe of the dam. This gravel pocket apparently had been overlooked during construction, although it reached for a number of feet under the toe of the dam. Fortunately, this dam had been built curved in plan so that no direct failure occurred and the damage could be repaired after the flood.

Furthermore, it is well recognized that many reservoirs will fill up with débris and silt after a period of years. Such solid matter will then, of course, exert additional pressure against the dam, unless adequate provisions are made to keep the silt clear from the dam. In many cases this may be rather difficult, or it may be neglected entirely. In the case of a long, straight gravity dam, such conditions may lead in time to a serious menace, particularly in view of the small factor of safety of such a structure.

The safety factor of a dam therefore should not only cover the possibilities of increases in direct load, as water pressure, ice thrust, etc., but it should also take care of such uncertainties as may occur during construction, or later, such as faulty construction, uncertainties in the foundation, pressure from silt, etc. The writer believes that, taking into consideration all these possibilities, the factor of safety of straight gravity dams is so small that it is most desirable to have this safety factor increased. The writer's investigations with regard to cylinder-hoof action were made particularly with this object in view.

On the other hand, if it is recognized that gravity dams have a factor of safety of less than 2, there is no reason why it should be necessary to require for arch dams factors of safety of 6 or 8. If the stresses in an arch dam can be figured with a reasonable degree of accuracy, it should also be permitted to use higher unit stresses, say, about 500 lb. per sq. in. at the present time, and from 600 to 800 lb. per sq. in. as soon as future tests have established a standard theory of arch dam design. Such higher unit stresses, which correspond to factors of safety of 3 or 4 with regard to the crushing strength of good concrete, would permit the building of arch dams comparatively thin so as to make steel reinforcement economically feasible. An arch dam for which the maximum unit stresses are, say, 600 lb. per sq. in. and in which all tension is taken care of by sufficient steel reinforcement so that no open cracks will occur either from temperature or shrinkage deformations, or cantilever action, etc., is undoubtedly safer than another dam for the same local conditions and in which the "cylinder" stresses are 300 lb. per sq. in., but which has developed open cracks. An alternating transmission of 300 lb. per sq. in. through irregularly broken surfaces is no doubt more dangerous for any concrete structure than twice that stress in reinforced concrete.

Mr. Hazen's remarks with regard to the amazing strength of masonry arches are fully appreciated. The fact that no evidences of moving, cracking, and spalling have ever been observed on arch dams may find its explanation by the probability that in no arch dam has the ultimate crushing strength of the material been reached at any point. In all the arch dams which the writer investigated the theoretical maximum unit stresses were seldom found to be over 1 000 lb. per sq. in. Those

stresses are therefore, although rather high, still far from the crushing strength of good concrete.

Mr. Jorgensen believes that "before any radical improvements can be introduced [in the design of arch dams] the elastic theory itself as applied to arch dams must be considerably modified". His principal reasons for this are that "if a time interval of 14 days or more is interposed between the occurrence of no stress and a high stress, the modulus of elasticity will have changed in such a way that the concrete will either have stretched about three times or have compressed about three times as much as it would have if the same stress had come on fairly suddenly, as on a bridge."

It is well known that concrete retains a certain degree of plasticity for many years. Consequently, its modulus of elasticity varies somewhat with age, and also it is not quite the same for high stresses as for low ones. However, the difference is comparatively small and for low unit stresses, between 0 and 400 lb. per sq. in., which are supposed to be the limits in present arch dams before failure in tension occurs, the modulus of elasticity is practically constant* for concrete more than three months old. This is one of the fundamental and standard assumptions of modern concrete design, and a value of $E = 2\,000\,000$ lb. per sq. in. is recommended by the Joint Committee on Concrete and Reinforced Concrete for practically all concrete construction. As the Joint Committee represents the highest authority on concrete in the United States, its recommendations certainly ought to be followed also by the designers of concrete dams.

The tests upon which Mr. Jorgensen bases his claims for a modification of the elastic theory as applied to dams were made either on reinforced concrete structures or in the laboratory only, and they can hardly be assumed without further proof as representing the conditions existing in large masonry dams. In no case can these tests be taken as sufficient evidence that for the small unit stresses occurring in arch dams a three-fold compression or stretching would occur before the concrete fails in bending on the tension side. Even good concrete has a maximum tensile strength of only 150 to 200 lb. per sq. in. For dams, the bond between masonry and foundations and in the joints between successive days' work is generally still less than this, and may eventually be zero. At the time when, according to the writer's calculations, the cantilever action in an arch dam begins to be destroyed, the tension at the up-stream side is therefore between the limits of 0 and 200 lb. per sq. in., and the compression at the toe probably between 200 and 400 lb. per sq. in. For instance, in the Lake Spaulding Dam the compression at the toe was about 190 lb. per sq. in. at the time tension started to develop at the heel. A deflection of ten times the amount which is obtained by standard calculations and based on assumptions recommended by the Joint Committee can therefore hardly be explained by the action of the "time factor", as Mr. Jorgensen apparently believes.

A pushing of the arch of the Lake Spaulding Dam in an up-stream direction by the chemical heat is hardly conceivable. The maximum temperature of hardening cement is developed a short time after the concrete is poured, and afterward the temperature falls constantly until it has reached normal. When the zero mark for the deflections was set at the Lake Spaulding Dam, all the concrete

* G. A. Hool, "Reinforced Concrete Construction," First Edition, Vol. I, p. 32.

had been in place for a considerable length of time, and in the lower portions of the dam for many months. Consequently, it had passed by far the maximum setting temperature, as evidenced also by the opening of the contraction joints and by cracks between these points. Still, Mr. Jorgensen is of the opinion that the chemical heat had pushed the highest arch in an up-stream direction for about 2½ in. This would have required an increase of the arch temperature above the closing temperature of about 60° Fahr., which is hardly possible, inasmuch as the concrete already had cracked, due to shrinkage and a lowering of temperature.

The remarkable thing about the Lake Spaulding Dam is, therefore, that this dam has deflected about ten times as much as calculations, based on assumptions recommended by the Joint Committee on Concrete and Reinforced Concrete, would show to have been possible before cracks would be expected. Possibly the fact that the concrete used for this dam had a breaking strength of only 900 lb. per sq. in. at the age of 28 days will throw additional light on the statical conditions of this dam, which acted so differently from other structures of similar dimensions. For instance, the Salmon Creek Dam in Alaska deflected less than 1 in.*, although this dam is of much thinner cross-section. The radius, however, is somewhat smaller, but the difference of the deflections of the two dams must arouse suspicion.

Mr. Jorgensen's assertion† that in his opinion, based on experience, "in order to make any assumption hold good for a curved wall in a canyon acting like an arch, it is necessary to fill the contraction joints with grout or concrete during cold weather, with reservoir empty" is rather startling. Thus, also, the assumptions of the cylinder theory would not hold good for all dams which have not been grouted. The danger of grouting narrow cracks has been emphasized by Mr. Wiley, and his arguments speak for themselves. On the other hand, it will undoubtedly be of advantage for the purpose of offsetting the contraction from shrinkage and a part of the seasonal drop of temperature to close arch dams during the colder season of the year. By far the best and safest way to do this consists in leaving open a number of vertical slots, say 18 to 24 in. in width, and closing them with well spaded concrete during the colder season. However, this ought to be done very judiciously, and particularly not at the time of the seasonal temperature minimum, because in some later summer, when the reservoir might happen to be nearly empty, the arches would be pushed in an up-stream direction and serious stresses in both cantilever and arches might result.

It is the opinion of the writer that all slender arch dams ought to be reinforced by steel bars to take care of all tension stresses. Swedish engineers recognized this necessity some time ago, as evidenced by the construction of an arch dam reinforced according to the theory of the elastic arch.‡

Mr. Jorgensen claims that in multiple-arch dams "no cracks ever occurred, the time interval between the occurrence of low stress and high stress was simply long enough for the concrete to stretch additionally without cracking." This can hardly be so, because the daily variations of temperature in thin arches when the water stands low in the reservoir are in many localities high enough to produce

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 321.

† In the discussion of Mr. Smith's paper on "Arched Dams", *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2083.

‡ *Journal of Electricity*, July 15th, 1920, p. 79.

hair cracks in the tension zones of the deformed arches. Since the arches generally are reinforced, these tension cracks are well distributed and hardly visible to the eye, as in the case of tension cracks which occur at the bottom of all reinforced slabs and beams even under ordinary load conditions. Many of the existing multiple-arch dams are only slightly reinforced. There exists, therefore, the danger that the steel is stressed beyond its elastic limit. The arches of multiple-arch dams are generally fixed at the buttresses and the theory of the elastic arch gives the stresses as accurately as in bridges.

Mr. Jorgensen also claims that an arch dam theory, to be complete, "must include the action of Poisson's ratio, shear, and beam action, which are as important as the cantilever action." Mr. Smith's theory includes shear, while by the writer's method it is neglected, and rightfully so, as shown by the results of Mr. Cain's comparison of the two methods, which give practically the same results for shear and no shear.

The action of Poisson's ratio or lateral expansion in dams of less than 200 ft. in height is largely or entirely offset by the shrinkage of the concrete, as is shown clearly by the occasional opening of contraction cracks. On the other hand, if lateral expansion should really occur in very high dams in more than a negligible degree, such expansion would evidently affect only the lower parts where cantilever action prevails. Equation (13) permits the calculation of deflections, D, due to the action of Poisson's ratio after the corresponding arch stress, f_c, has been determined. Calculations will show that for most arch dams the influence of Poisson's ratio may be neglected as being very small, if not entirely offset by the shrinkage of the concrete.

"Curved beam action" in arch dams is of such a similar nature to arch action that with very close approximation the theory of the curved beam in an arch dam is covered by the arch theory. The arches (often wedges) in the lower parts of arch dams are stiffer and deflect less than those same dam slices would if they were considered as curved beams. On the other hand, the cantilevers deflect still less at the bottom than the arches. The load is naturally supported in proportion to the stiffness of the foregoing three elements. Consequently, the cantilever takes practically all the load near the bottom.

The writer's method of arch dam design, therefore, takes care of all considerations involving shear, Poisson's ratio, and beam action.

It has further been claimed that the elasticity of the bed-rock on which a dam is built would permit larger deflections for both the cantilever and the arches than the writer assumed for his theory. In general, an arch dam is "keyed" from 3 to 5 ft. into the solid rock. The shearing and bending stresses transmitted from the dam body to the foundation act, therefore, on a body of tremendously increased cross-section as compared to the thickness of the dam. The unit compression in the masonry just above the foundation may be several hundred pounds per square inch at the toe of a large dam, but this is distributed within a short depth in the bed-rock over a largely increased area, because deformations of the bed-rock will be transmitted to adjacent parts in proportion to the elastic deformations. The unit stresses in the bed-rock decrease very rapidly, therefore, toward the interior of the rock so that the elastic deformation of the rock foundation of any dam is undoubtedly so small as to be negligible for practical purposes.

For arch bridges on a rock foundation the maximum unit stresses between arch and rock (600 to 800 lb. per sq. in.) are in general much higher than in arch dams. Nevertheless, such bridge arches are considered as absolutely fixed at the ends, and theory and practice agree very well, as shown by many tests. For these reasons, the writer is of the opinion that the small elastic deformations of the bed-rock are without any material influence on the deflections of arch dams.

Mr. Kesner has himself answered his question why the analysis of the pressures in arch dams, as published by the U. S. Reclamation Service and also on page 474 of the edition of "Irrigation Engineering", by Davis and Wilson, gives only approximate results which differ considerably from those obtained by the writer's method. "This error is inherent in the formula, since it considers the pressure proportion, K_g, at any level to exist also below that level, giving a triangular distribution of load on the cantilever element."

The process of combining the formula for the arch deflections with the one for the cantilever deflections is leading to incorrect results, in so far as the cantilever deflection formula would have to be changed for other than triangular load distribution. As shown by Fig. 14, the resulting cantilever load is, in fact, not of triangular form, so that the final formula for K_a is only an approximation. The cantilever and arch deflection lines for the load distribution, as shown by Fig. 14, do not coincide with each other, as may easily be ascertained by the graphical construction. This is at once evident for a point at the crest; the arch deflection is zero, as there is no water pressure acting on the highest arch, while the cantilever curve undoubtedly would show a deflection at the crest due to the load on the cantilever in the lower portions of the dam.

Mr. Neubauer compares the writer's graphical method with Mr. Smith's analytical one from the purely mathematical point of view. Both methods lead to practically the same results, as shown by Mr. Cain's discussion. Consequently, each engineer may use that method which suits him best.

The writer agrees with Mr. Parker that his paper is to be taken mainly as marking a "point of departure" for a much further advance to be made in arch dam design. Such further advance will be made possible particularly by stress and deflection measurements of existing dams. With regard to the Crowley Creek Dam as described by Mr. Parker, the writer joins in saying: "This example cannot fail to cause wonderment as to what load of water pressure a well built, true, and symmetrical thin arch really could safely carry." The construction of this dam undoubtedly marks another point of departure in dam construction, although apparently more from a practical than a theoretical point of view.

Mr. Wiley's figures with regard to the temperature changes in the Arrowrock Dam are of particular interest, in view both of Mr. Smith's and the writer's investigations as to the stresses in arch dams due to variations in the temperature of the dam body. However, it is probable that in slender arch dams the variation of temperature follows different laws than in the large masses of a gravity dam. Only actual measurements can establish a definite rule for arch dams.

There is no inconsistency in the writer's treatment of the division of the load between cantilever and arches. Case 1 refers particularly to curved dams of full gravity section and Case 2 to rather slender arch dams. If in a dam of the first type the temperature happens just to equal the closing temperature of the arches,

then, of course, both cantilever and arches share in the load. At the time of low temperature the arches are shortened and consequently they may have the tendency to deflect more than the cantilever under full water pressure. The example of the Lake Cheesman Dam clearly shows that such conditions may occur in practically all gravity dams.

The writer cannot agree with Mr. Wiley that if the cantilever is tested for stresses in both faces, very high dams would closely approach the dimensions of the gravity type. They may do so, of course; on the other hand, also, very high arch dams are feasible by allowing higher working stresses and making the cantilever, particularly in the lower portions, very slender and flexible.

Also, the writer's statement is correct "* * * that in the common type of arched dam, for which the radius of the center line of the horizontal arch slices is more or less the same, the deflections, D, at the crowns are approximately the same for the lowest arches as for the highest ones, irrespective of the length, provided the unit stresses in all arches are the same." The proof for this assertion is given by Equation (13),

$$D = \frac{1.56}{E} R f_c$$

in which all values are constant for constant R and f_c, and therefore also the deflection, D.

The writer considers Dr. Ritter's method of obtaining the complete coincidence between cantilever and arch deflection lines in one single operation as of great advantage in many cases. Thus, all "trial and error" computations may be avoided and the final result will be the more accurate the greater the number of horizontal arch divisions assumed for the investigation. Dr. Ritter's suggestion to represent the deformations resulting from the temperature and shrinkage as if they were produced by a certain load is correct, and it will lead to a more accurate determination of the resulting stresses.

The writer agrees with Mr. Wegmann that it is rather doubtful whether the modulus of elasticity in a large dam built of rubble concrete, may be determined very accurately. In many slender arch dams, however, the dimensions approach very closely those of other concrete structures such as bridges, etc., for which many tests have shown a fair coincidence between calculated and measured deflections and stresses. The writer's method of calculating the stresses in arch dams leads to more accurate results the less the thickness of the arch as compared to height and span. For very thick dams in narrow canyons it is admittedly only an approximation.

The tests cited by Mr. Wernecke, which show a certain "weakness" of the cylinder formula also for full cylinders under external pressure, are of particular interest in connection with the problem of arch dams. They also emphasize still more the great desirability of testing arch dams.

Referring again to the assumption that the bed-rock is unyielding, it is a fact which may be proved both by theory as well as by experiment that a structural member may be considered as "fixed" at the point where it joins another body of much larger cross-section. Although undoubtedly some elastic deformations

occur in the bed-rock on which a dam is built, such deformations are so small under the existing unit stresses of only a few hundred pounds per square inch that they may be neglected for all practical purposes. The deflections on the Lake Spaulding Dam were measured at the center of the arch crown and, therefore, owing to the unsymmetrical profile of the dam site, practically over the bed-rock foundation adjacent to the concrete base. The elasticity of this greatly enlarged concrete base, therefore, cannot have been the reason for the excessive deflection of this dam.

The writer does not care to argue the points brought up by Mr. Jakobsen. His discussion is a sad example of destructive criticism without even an attempt to suggest how to do better. His assertions strike at the root of all present standard assumptions for concrete design as recommended by the Joint Committee on Concrete and Reinforced Concrete, and as used by the writer throughout. On the basis of his own assumptions and some extreme test results, Mr. Jakobsen claims that all the writer's calculations are not only absolutely wrong and useless, but even harmful. Directly and indirectly, he also makes the same claim in regard to all the studies relating to arch dams which have been made by such eminent scientists as Messrs. Woodard, Shirreffs, Cain, B. A. Smith, and many others who have devoted much earnest effort to improving the design of arch dams by also taking into account cantilever action. Mr. Jakobsen tries to ridicule every past and present effort and to discourage also future attempts for progress in the methods of design of arch dams and concludes that the Rankine (cylinder) formula is the most satisfactory.

Furthermore, Mr. Jakobsen replied to the writer, who inquired about deflection measurements of the Kerckhoff Dam, that he did "not know of any having been made, and if they had been made, they would have been quite worthless for any purpose of stress calculation." Mr. Jakobsen thus, in the very important matter of the desirability of deflection measurements, announces his position in regard to possible future improvements in arch dam design.

Mr. Miller's criticism with regard to the accuracy of the writer's Equation (15*a*) is justified for an extreme case such as that cited, for which the thickness of the arch is equal to its rise. Such conditions, however, occur only in the lower parts of arch dams where cantilever action will prevail, so that the secondary arch stresses are of little importance. However, if it is also desired to obtain a maximum degree of accuracy for very extreme cases, it might be advisable to apply for every arch slice the exact theory of the elastic arch without using any of the simplifications and short-cuts suggested by the writer. As stated at several points, his method of designing arch dams is the more correct the thinner the arches as compared to rise and length. Toward the crest of arch dams, where the influence of temperature changes is of particular importance, the writer's approximate formulas will furnish fairly accurate and reliable results for true arch dams.

Mr. Luiggi's discussion is very welcome as a contribution coming from a country where in the last few years the design and construction of arch dams has made most remarkable progress. It is to be hoped that Mr. Luiggi will soon describe in more detail the design and the more important features of the 200-ft. multiple-arch dam which he mentions in his discussion. Similar structures of about equal height are contemplated also in the western United States. A description of the

design and European construction methods for multiple-arch dams of such height would certainly be of great benefit to American engineers.

In conclusion, the writer wishes to state that it was most gratifying to note the general interest shown in the problem of arch dams, as evidenced by the participation in the discussion of his paper. The correctness of his conclusion that probably all existing arch dams of considerably reduced gravity section, and which have not been anchored to the rock foundation, have failed in cantilever action, has been doubted by many members. Such doubts, however, have been expressed only as matters of opinion, and were not substantiated either by calculations or tests.

Arch dams of unprecedented size, from 350 to 450 ft. high, are contemplated for the near future. It is to be hoped that authoritative measurements and tests are soon to be made to establish beyond any doubt the standards for the safe design of such structures, instead of having, perhaps, the actual failure of an arch dam mark the limit of the application of the cylinder formula.

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Paper No. 1464

FEDERATED ENGINEERING SOCIETIES OF AMERICA

ADDRESS AT THE ANNUAL CONVENTION IN PORTLAND, ORE., AUGUST 10TH, 1920.

BY ARTHUR P. DAVIS, PRESIDENT AM. SOC. C. E.

The demand for active participation of all engineers in civic and other work for the general benefit of mankind and the advancement of the Engineering Profession has been growing for many years, and is greater to-day than ever before. Various plans for accomplishing this purpose are suggested, and the pressure has been such that the American Society of Civil Engineers and the other Founder Societies have each taken a more or less active part in such work.

Recognizing the necessity of combining efforts in this line, and also the desirability of somewhat separating such activities from the technical functions of these societies, Engineering Council was formed with the idea that it would represent the four Founder Societies and gradually add to this number through powers conferred upon it for that purpose. It has, in the three years of its existence, added the American Society for Testing Materials and the American Railway Engineering Association, but is still far from embracing all of the Engineering Societies of the country. It has some fundamental weaknesses. It lacks in representative character and democracy, and is without power to raise funds, but must depend upon voluntary contributions from its constituent societies and others. Experience has shown that these deficiencies can be best remedied by substituting a league or federation designed to include all of the Engineering Societies, National, State, regional, and local. A constitution to this effect was adopted at the convention held in Washington in June. The federation does not provide for any individual members, but is merely an instrument for combining the efforts of existing societies by forming them into a federation with definite powers and provided with sufficient funds to accomplish its purpose.

The unanimity of the conference was practically complete, with the exception of the American Association of Engineers, a recent organization which has been rapidly growing as the result of active and persistent drives for membership. It fills a niche among Engineering Societies by admitting large numbers of men working in engineering lines, but without the necessary qualifications to be admitted to full membership in the Founder Societies. It has been quite active

and has accomplished good work. It can accomplish still more by continuing its efforts along right lines. But its ambition to be an all-inclusive society, representing all grades of the Profession, can scarcely be realized, and seems unreasonable to expect; and even though it should achieve a greater success than is expected along this line, its prestige is not such as to give it standing comparable to the Federation of the four Founder Societies and other large organizations with long standing, prestige and high standards.

In spite of the unanimity of the Washington Convention, there is still a small but very active minority in the American Society of Civil Engineers that opposes this Federation and professes to oppose any participation of the American Society of Civil Engineers in civic or other welfare work. Due to the peculiarities of an antiquated Constitution, and to a particularly effective organization, this minority has an influence within the Society far beyond its numerical strength. Up to date it has, in the main, governed the policies of the Society, yielding to pressure only when this became irresistible. It opposed the junction of our Society with the other Founder Societies in the Engineering Societies Building, and opposed the formation of United Engineering Society, and successively each and every step tending to co-ordination and co-operation, of the Founder Societies. In spite of the overwhelming majorities in the recent vote upon Questions 1 and 2 touching the principles involved in the federation, and the recent progressive majority shown by the Committee on Nominations, the conservative element is said to be contemplating an independent or insurgent ticket, supported of course by the usual arguments of caution and conservatism, and utterly ignoring the blunders of the past which such alleged caution has involved.

The real issue in the contest now going on, and especially in the election of Directors which will take place next fall, is whether or not the Society is to take its proper place among human activities, or gradually decay and become an obstruction to progress, pretending to fill a niche which it does not fill, and wasting the prestige and history of this glorious Society which ought to be devoted to some end more useful than individual benefits to a small circle of its membership.

In opposing progressive tendencies the advice is offered that the Society restrict its field to technical matters, and leave civic activities to others.

This advice would come with more convincing force were it not for the fact that those who offer it have controlled the policies of the Society for the past twenty years, and have so neglected the technical field that the demand for technical standards and specifications, which it should have furnished, could be met only by the organization of new societies, such as the American Railway Engineering Association, and the American Society for Testing Materials, whose activities have been largely directed by enterprising and public spirited members of this Society, who could and would have performed the same services through committees of the American Society of Civil Engineers had they been given proper encouragement and facilities. After having frittered away the opportunities for usefulness and growth in the technical field, it now ill becomes these false leaders to advise us to confine ourselves to the restricted remainder of that field.

The argument usually given is that civic or welfare work might violate the dignity of the Society. We might advocate mistaken policies, and afterward be

obliged to modify these. This is the same excuse given for shirking its technical duties in the past. It was feared that if our Society adopted and uttered standard specifications, these might be found imperfect or become obsolete, and the necessary changes would reflect upon its dignity. Such argument, if relied upon, would effectually block all attempts at usefuless in any field. It has caused untold damage to this Society and the Profession, and should be discarded forever. We should use due diligence and care to avoid errors, but the possibilities of error should not be made an excuse for inactivity. Whatever the real issues back of the conservative advice, its tendencies are unmistakable. The demand for broader activities is from the country at large, and the opposition is confined mainly to three or four of the large cities. These, with their compact and well organized membership, have been able in the past, with the help of our antiquated Constitution, to control the main points of policy of the Society, and to perpetuate themselves in power. This domination is threatened by the growth of membership in the rural districts which would be stimulated by the measures proposed by the Committee on Development, especially those measures designed to increase the growth and influence of Local Sections, and the expansion of activities in accordance with their demands. Every measure that is popular with the membership outside the great cities will add to its growth, and proportionately tend to terminate the centralized control that has hung like a millstone about the neck of the Society for so long, retarding its growth, restricting its usefulness, and threatening it with dry-rot.

Two Distinct Issues.

There are thus two distinct issues involved in the pending measures and the coming election of officers. These issues are "progress *versus* stagnation" and "democracy *versus* centralized domination". These issues are largely interdependent, and upon their determination depends the future usefulness and growth of the Society. Their importance can hardly be overestimated.

Measures of great importance to the world, to our country, and to our Profession demand immediate and vigorous attention, and it is imperative that they have behind them the unified efforts of all the organized engineers of the country, and all the prestige that the greatest, the oldest, and the most eminent societies can offer.

Among these measures are the unification of the engineering work of the Government under one department with a technical head. This will promote efficiency and eliminate waste, and what is more important, will serve as a precedent for the reorganization of other departments along similar lines of homogeneity, efficiency, and economy.

Other important measures pressing for attention are the laws proposed in many States for the licensing of engineers. This policy is steadily progressing. In some States this movement is doubtless inspired by a desire to shut out the competition of engineers from other States, a most destructive policy, and unless some recognized authority, like the Federation, backed by both the leaders and the rank and file of engineers, takes a strong hand, we may find ourselves burdened by a mass of pernicious laws that will work untold harm before they can be swept away or otherwise remedied. Properly directed, however, this movement has beneficial possibilities.

No one has yet explained the origin of life upon this globe, but it seems certain that the world began and developed for millions of years without any life at all. Geology shows that when life appeared it was in the most primitive forms. Millions upon millions of years were required in a biological evolution from the primitive protoplasm to the animal we call man. Even after man appeared, it required many thousands of years to develop from the Stone Age through all the intermediate stages of savagery and barbarism to civilization, so-called. Within historical times, several thousand years rolled by during which the advance in human intelligence was, in any limited time, almost infinitesimal. During extended periods, human progress appeared to be at a standstill, or went backward, and of the past 5 000 years of which we have some record, not more than 500, or the last one-tenth, show much tendency to progress. In fact, even in that time, there have been times of retrogression, and the progress was imperceptible for long periods.

This brief reference to the history of evolution is merely to remind us of the numerous times that human progress has been retarded or turned backward, and that a similar fate may befall the world in the present wonderful era unless the intelligence and ability of the time make active study of conditions and tendencies, and devote themselves diligently to the correction of abuses and the establishment of justice.

Scientific Investigation a Recent Development.

Not until the last century was any considerable impetus given to scientific investigation, and the application of its discoveries to the uses of man. But during that century there has been progress along material lines, constantly accelerating, and as we look back we find that the great acceleration has apparently been increasing. We have arrived at the point, in fact, where the problem is not the problem of production, but of distribution. Even now when we talk of the high cost of living and find that many things are scarce, the reason is not in the lack of productive power but in an unnecessary and abnormal indulgence on the part of some to the deprivation of others, and other abnormal conditions, and while much is to be desired in the way of accelerating and cheapening production, these arts are so far advanced and their certainty of improvement is so great, that they may be dismissed as secondary to the greater need for a more equitable distribution of the things produced.

We find great disparity between the rewards received by men, and this disparity is by no means in proportion to ability or to any other circumstance which contributed to production. In other words, it is not the man who works the hardest who becomes the richest. It is not he who produces most that receives most. There are problems involved in distribution which have not been solved. Social and economic progress has not kept pace with material progress, and the time has come when the discrepancy must be bridged over, for further increase of production unless accompanied by a more reasonable distribution is likely to become a menace rather than a blessing. It is to the solution of these problems that the leaders of men must give their attention if civilization is to be saved, instead of destroyed, as all past civilizations have been.

I have neither the time, the ability, nor the assurance to attempt a solution at this time of any of the great problems presented, but will call attention to typical illustrations of what these problems are.

Expansion of Government Activities.

The great changes caused by the World War and the accompanying upheavals have led irresistibly to larger functions of Government. It has become necessary to increase those functions to prevent great injustice and suffering. The control of our transportation lines and other public utilities, once criticized and condemned, is no longer questioned. It is admitted by all to be inevitable and unavoidable. It has even become necessary to control the price of food and of other commodities, and the pressure is strong and the argument almost irresistible for extending this control in many other lines. Rent commissions have been established, arbitrarily fixing the price which the owner may charge for his property, his title to which has never been questioned, except in this indirect manner.

The need for these vast expansions of Governmental activities admonishes us that they must be restricted to those cases where such expansion is absolutely necessary. Many who years ago advocated the public ownership of all public utilities such as railroads, gas-works, water-works, and power plants, have in the last five years changed their minds. For one thing, public control through judicial commissions has proved more successful than anticipated, due mainly to an awakened public consciousness, or at least a more enlightened public selfishness. But fully as potent a reason for this changed attitude has been the alarming demands upon the Government for the expansion of its functions in other lines, and it becomes important therefore to confine its functions to those which are inevitable or absolutely necessary. I am therefore in hearty sympathy with many who are now opposing the public ownership of railroads and power plants and commercial shipping. But while agreeing with the conclusions, I must strongly condemn the principal reason which they advance for this. It is common to hear intelligent men whose motives cannot be questioned, basing their opposition to Government ownership of railroads on an argument which is condensed into a statement or opinion that "Government operations cannot possibly be as efficient or as economical as those under corporate control." One objection to such a statement is that it is untrue. It has been proved over and over that this is not necessarily the case.

While admitting that there are, and always have been, abuses and defects in our postal system, who would be so bold as to assert, or if asserted, who would believe the assertion that our mails would be carried more cheaply, promptly, or economically by private enterprise than they are at present.

The public activity with which I am most familiar is the United States Reclamation Service. No claim is made for infallibility or super-human wisdom in the conduct of that service, but its principal officers have always striven for the highest efficiency and economy. It has not been without its mistakes; but results as a whole have proven the fallacy of the theory that the Government cannot do things economically and well, and to prove this let us submit to the judgment of men who know and have such interest that they must choose.

In the State of Idaho the Government has carried out two of its largest projects, including four storage reservoirs, thousands of miles of canals, large power and pumping plants, and the irrigation of nearly 300 000 acres of land. These are works which affect the people in their homes and their pockets. If they involve faults, these stand out in bold relief and the public is sure to hear of them. In the

same State on the Upper Snake River are a large number of private canals taking water from that river, which have plently of water in the spring and early summer months, but after the snows are melted the river declines and shortage of water supply exists during the late summer and autumn on these private canal systems. The farmers under those canals are desirous of an additional water supply which can be provided by the construction of a large storage reservoir at American Falls. This reservoir, if constructed, will be the largest irrigation reservoir in the world, costing $15 000 000. The Reclamation Service has no funds available for this construction, but was requested to make the necessary investigations to determine its feasibility and cost. The farmers desiring a water supply organized, and about forty different canal companies joined the organization. A questionnaire was circulated stating that it was proposed that this reservoir be constructed for the interested canal companies which should advance the money for the purpose as needed. Each company was requested to indicate the number of acre-feet of storage capacity desired under these terms, and also to express a preference as to who should undertake the construction of the reservoirs with the money which the canal companies were required to advance. Over 90% of these hard-fisted farmers voted that their money should be used in the construction of the reservoir by the United States Reclamation Service. Plenty of contracting companies at Salt Lake, Boise, Portland, and other accessible points would be glad to undertake the job; but these farmers have seen the work done by the Government; they are acquainted with its officers, and form their own judgment as to whom the greatest dependence could be placed upon, and almost unanimously chose the United States.

So far as it goes, this is the evidence of these farmers that they believe the Government can do work as efficiently, as cheaply, as quickly, and as well as private enterprise.

Efficiency of Government Work.

So much for the fallacy of the theory that Government work cannot be done efficiently. It may be admitted that frequently great extravagance, waste, mismanagement, politics, and even corruption exist in the transaction of the public business. These are faults to be eliminated and not condoned by the apology that honest, efficient service by the Government is impossible. That is the apology of every spoilsman, grafter, and incompetent in the service. He or his friends wave criticism aside with the testimony of our greatest statesmen that Government work cannot be done economically. It is a false, pernicious principle, calculated to perpetuate the very abuses which give it plausibility, and every one who desires to improve the public service should condemn such an argument and challenge such a statement.

Government work can be done as cheaply and efficiently as any—in some respects more so. The Government has better credit than almost any corporation. Due to the supposed honor attached, it is possible to secure a given talent in the Government service more cheaply than any other. No corporation can tempt a Government expert from his job without greatly increasing his salary or emoluments. With proper business management the Government can purchase cheaper than any one else because of its superior credit and stability. It is true, and must be admitted, that these results do not always obtain. Through unwise laws, red tape abounds.

Through insufficient salary, incompetence sometimes occurs, and I am sorry to say it is still a fact that politics sometimes interferes with efficiency; but these are all faults that can and must be remedied. For any one to say it cannot be done is to discourage the attempt and to apologize for the offenders. Where Government operations are not as economical and as efficient as other activities it is somebody's fault. The fault should be located and removed. It should not be condoned by the statement that it is inevitable. Political favoritism, or any other kind of favoritism, should be eliminated from public service. This can be done only by force of public opinion, and to this end public opinion should be stimulated and when aroused should be exerted. Criticisms of faithful public officers should be condemned and denounced as they deserve, but criticism of the incompetent or the grafter is a public duty, and what kind of graft is so pernicious or damaging as the distribution of public personal favoritism?

The grafter who falsifies a voucher and steals a hundred dollars from the Government is deserving of criminal punishment. But he is a venial offender compared to the politician who appoints an incompetent favorite to a high office where his disastrous management may cause the loss of millions of dollars and perhaps of human lives; and he who paralyzes progress by spreading the fatalistic doctrine that public service cannot be improved, but is destined forever to be wasteful, extravagant, and incompetent, does infinitely more harm than the petty grafter whom we punish.

A New Menacing Tendency.

One of the most menacing tendencies of recent years is that which is crudely described in common parlance as Bolshevism. This term doubtless has a fairly accurate scientific meaning which differs somewhat from the common conception of the word, but the general understanding of this term is embraced in the expression of contempt for the rights of men, and especially their right to property. This contempt was founded originally upon unjust laws which have been overthrown, and have not been replaced by anything better. History of the rise of Bolshevism in the countries now suffering from its palsy is the history of oppression and of contempt for human rights almost as gross and brutal as the communistic theories of Bolshevism, and from this a lesson may be drawn.

If we wish to inculcate respect for human rights, and especially rights to property, we must ourselves respect those rights. Careful observation teaches us that the same causes of contempt for the rights of property which led to the overthrow of all such rights in Russia are existent in this country and that not only the so-called radicals, but some of the most conservative thought of the country is spreading doctrines which if accepted must eventually lead to utter disregard of the rights of property. I recall in a leading magazine about a year ago an article by so eminent an author as Dr. Lyman Abbott, under the heading "Popular Fallacies". The general trend of the article was an effort to prove that there are no natural rights of property. This doctrine is summed up in one of his paragraphs as follows:

"Neither making nor buying gives us, according to a standard of absolute justice, a perfect title to property. Such title as we possess is given to us by the laws and the customs of society."

This dictum was supported by columns of argument tending to prove that our standards of property owning are purely conventional and artificial and that there is no natural basis of right to property. The Bolshevist or the Communist who is desirous of substituting some arbitrary scheme of his own which may deny many or all of our present property rights, may quote the aforesaid doctrine and if it is sound no moral objections can be made to his doctrine of equal division of all property irrespective of any rights that may be asserted by the owners or producers of wealth.

This doctrine not only destroys all ground for defending present claims to property rights or any future claims or standards that might be established, but carried to its logical conclusion, also destroys all rights of personal liberty; for if the law can with impunity take away the products of our labor, it can in effect enslave us, and this is in fact the most important result of any system of slavery.

One of the poems of James Russell Lowell, an attack upon chattel slavery, contains the following:

"Slavery ain't o'nary color,
'Taint the skin that makes it wuss.
All it cares for in a feller,
Is to make him fill its puss."

Those who believe there is a natural right to property base such a belief on the natural right to liberty and the use of one's own faculties, and consequently to the results of the exertion of those faculties. Whether or not this is the basic principle of property rights, it is important that we discover some ethical basis of such rights and adhere strictly thereto, for if we cannot appeal to men on the basis of righteousness, on what basis shall we appeal? Certainly it is of the utmost importance that we determine an ethical basis for asserting the right of property, that we establish those rights upon that basis, and rigidly adhere thereto; for no citadel is invincible except the citadel of righteousness.

The contempt for human rights, and especially for property rights, has apparently been growing of late years without seeming to attract attention, and perhaps without the consciousness of the great majority of our citizenship. For example, we have seen a law passed under the operation of which more than $3 000 000 000 of paper money have been added to our circulating medium within five years, with the inevitable result that the value of all money has greatly depreciated and the cost of commodities has, on the average, nearly doubled.

This is wilder inflation than ever proposed by Greenbackers or Free Silverites in their palmiest days. To some extent wages have adjusted themselves to the fluctuating value of the dollar, but this adjustment has been more a matter of chance than of justice or of the operation of natural laws. The large body of intelligent workers has shared but little in the general advance, while the cost of all they consume has increased beyond all reason. This imposes great hardship upon such people, and among those people may be included the great mass of engineers. But these are matters that have been adjusting themselves, and can be made to adjust themselves by organization and further effort.

A more serious result of this unprecedented inflation is the fact that it operates to a great extent in the repudiation of existing contracts. The thousands of prudent, industrious citizens who have invested their savings in Government,

State, municipal, railroad, or other corporation bonds have had the value of their securities cut in half by being payable in dollars that are only half the value of the dollars invested.

Inflation not only robs creditors for the benefit of debtors under existing contracts, but by unsettling values, stimulates speculation and, inevitably, leads to concentration of wealth in a few hands, like the usual results of the gaming table. It substitutes the spirit of speculation for that of industry and is responsible for a large part of the alleged high cost of living which, due to this speculative spirit, has proceeded beyond the direct depreciation of the currency caused by its inflated volume.

This glaring and criminal robbery of one class of our citizens has gone on progressively with little complaint and less attention to complaint for several years, and is still in progress. It constitutes one of the most flagrant violations of the sanctity of contracts that this country has ever known, and no real attempt has been made to stop it. The law was passed and administered by one political party, and the other, now in power, has made no attempt to correct it. Not a bill has been introduced, so far as I am aware, to check the inflation, much less to correct the wrong already accomplished.

This is a non-partisan matter, and is one in which no specific organization of engineers ought to take sides, perhaps, but it illustrates the problems imposed upon all good citizenship, and one of the prime objects of a welfare organization should be to invoke the interest of its members in public affairs of importance, such as this. The sentiment really needed is regard for the rights of others, which, properly interpreted, means an intelligent regard for our own rights.

As Lowell says:

> "New times demand new measures and new men;
> The world advances and in time outgrows
> The laws that in our fathers' day were best;
> And, doubtless, after us some purer scheme
> Will be shaped out by wiser men than we,
> Made wiser by the steady growth of truth.
> The time is ripe, and rotten-ripe, for change;
> Then let it come; I have no dread of what
> Is called for by the instinct of mankind.
> Nor think I that God's world would fall apart
> Because we tear a parchment more or less.
> Truth is eternal, but her effluence,
> With endless change, is fitted to the hour;
> Her mirror is turned forward, to reflect
> The promise of the future, not the past."

The spirit of the progressive movement is summarized in the term "Service through Co-operation." It means service to mankind, to our country, to our Profession, and to ourselves. It cannot reach its fullest expression without the active co-operation and influence of every engineering organization in the country. It would be little short of a crime for the American Society of Civil Engineers, with its influence and prestige, to hold aloof. As Bacon has it:

"I hold every man to be a debtor to his Profession, from the which as men of course do seek to receive countenance and profit, so ought they of duty to endeavor themselves, by way of amends, to be adept and ornament thereto."

The preservation of human rights and of the results of scientific progress in the present crisis will require the united efforts of all intelligent, patriotic citizens. In this category I place all engineers; and as they owe a debt of gratitude for the blessings of civilization which they enjoy, they must repay this debt by vigilance and activity in the preservation and improvement of these blessings. Such a programme will have its difficulties and its pitfalls, but these will not deter nor discourage worthy men in the pursuit of excellence. Rather, the dangers will serve as a stimulus to activity.

We cannot act effectively unless united, and by calmly reasoning together on pressing questions, guided by the pole star of righteousness, we can unite upon fundamental principles, and make our influence for good a power in the land, and only thus can we perform the duty we owe to ourselves, to our fellow men, and to posterity.

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Paper No. 1465

LARGER SHIPS, DEEPER HARBORS, AND BETTER DREDGES*

By A. W. Robinson,† M. Am. Soc. C. E.

With Discussion by Messrs. E. J. Dent, Walter S. Wheeler, George B. Massey, Walter Ferris, Charles Evan Fowler, J. Vipond Davies, Edward Burr, J. H. H. Muirhead, William J. Wilgus, Edward F. Haas, Friend P. Williams, Chandler Davis, and A. W. Robinson.

Synopsis.

The purpose of this paper is to call attention to the need of better dredging machinery to meet present-day conditions, and of deeper channels and harbors in order to obtain increased economy in marine transport, this being the end to which efficient dredging machines are the means. If present-day costs of labor, fuel, and material are to be met, a new standard of efficiency is necessary, and projects requiring dredging should now be reconsidered in the light of these increased costs, which should be offset as far as possible by improved and more efficient machines, as described in the paper.

Three conclusions appear from what follows: First, that increased economy in marine transport can be gained by larger units more efficiently utilized; second, that deeper channels and harbors are essential to such economy; and, third, that more modern and efficient dredging plant than has been used heretofore is now required.

These points are not new, indeed they are almost self-evident, but a discussion of them may lead to a better understanding of the means by which the desired ends can be accomplished.

The expansion of shipping will bring this country into touch with world-wide conditions, and must bring with it changes in present methods. In the matter of dredging machines and port equipment the United States is—with some exceptions—behind other nations, and there is no reason why there should not be developed a distinctively American practice better suited to present needs here

* Presented at the meeting of October 6th, 1920.

† Montreal, Que., Canada.

Recent events have brought home the importance of transportation both by sea and by land. When transportation breaks down or is overtaxed in any part, the whole system of civilization and industry is affected. The life of the nation is dependent on facility of interchange, both internal and with other nations.

In the development of the pathways of transportation, both by sea and land, dredging and excavating machines play an important part. They are the tools by which the engineer builds the railways, digs the canals, deepens the harbors, channels, and waterways, reclaims and fills in waste land, and reshapes the earth to his needs wherever necessary.

The fact that increased economy is obtained by increased size and capacity has caused a great advance in the size and power of water-borne vessels and in the rolling stock of railways. In railways, the limit has about been reached, owing to gauge of track and bridge and tunnel limits. In ships, there is no limit except the depth of harbors and channels and the means for supplying and transshipping cargo. Ships are everywhere restricted in size and economy by the depth of the harbors and channels where they must trade.

For internal waterways, heavy loads can be carried on light draft by increasing the area and number of units, but for ocean-going vessels deep draft is essential. This is not only for sea-worthiness and strength, but also for carrying capacity and economy of propulsion. A steamship with a capacity of 25 000 tons dead weight should have a normal draft of 38 ft. If because of insufficient channel depths she must go to sea at 30 ft., that 8 ft. of draft is equal to a reduction in her carrying capacity of about 7 500 tons. Such a vessel would be more seaworthy if loaded to her proper draft, and, furthermore, would carry that increased load at the same expense for crew and fixed charges and only a very little more propelling power. In a vessel of 50 000 tons the value of deep draft is still more marked. If these considerations were more widely understood, there would be fewer small and inefficient vessels built and more attention paid to developing the most economical ship and providing the necessary depth of water for its use.

During the shipping excitement consequent on the war, a great number of small vessels were built for the emergency without counting the cost, and the question of larger and more efficient vessels is now receiving close attention. Even yet there is not a sufficient appreciation of the imperative need for a radical reorganization of shipping methods, nor a due sense of the wastefulness of the present system, or lack of system, in assembling, loading, and discharging large cargoes.

In a recent paper on "Economy in Ocean Transportation",* the writer pointed out that economy in the generation and use of propelling power of ships is now highly developed, that the gains that can be made in that direction are comparatively small, and that it is in the direction of larger units more efficiently utilized that one must look in order to obtain a greater and more economical carrying capacity. Where a saving of 2 to 5% can be made by refinements in the design of propelling machinery, there are many cases where the cost can be cut in two by larger units more efficiently used.

Thus, a vessel of 2 500 tons dead weight consumes 10.6 lb. of coal per 100 ton-miles at 11 knots, while a vessel of 25 000 tons consumes only 2.9 lb. per 100

* *Journal*, Eng. Inst. of Canada, February, 1919.

ton-miles at the same speed. In other words, a given amount of cargo is carried in the larger vessel with 27% of the fuel, a saving of 73%, with intermediate sizes in proportion. Ships of even larger size would be available and still greater economy secured as soon as the necessary depth of channels is obtained and proper provision made for the assembling and rapid handling of large cargoes.

Efficient Draft Depths.

Naval authorities state that the most efficient draft for a ship 1 000 ft. long is 57 ft. 6 in., and that it is not unreasonable to predict that within 20 or 30 years a depth of harbor of 60 ft. could be profitably used. British harbor authorities at the present time have expressed a preference for at least 45 ft. as a minimum depth which harbor engineers should recommend and work for, because although at the present moment 40 ft. might be a satisfactory minimum working depth, the demand for shipping would grow so steadily as to make 45 ft. necessary before even this depth could be achieved.

Thus, it is clear that immense savings can be made, which will vitally affect the whole system of ocean transportation, as soon as the channels can be provided and the necessary terminal facilities co-ordinated to suit the most economical ship. The field of dredging plant, therefore, is intimately connected with the whole subject, for it is on modern and efficient tools that engineers must rely to deepen the channels which make economical water transport available.

As in railways economy in haulage has been obtained by increase of train loads and in marine transport by larger vessels, so in dredging plant larger and more powerful units should be utilized on all works of importance. The limit to the size of a dredge is only in the magnitude of the work it may have to do, to insure that it will be profitably used, and in the facilities for the disposal of the dredged material.

The greater depths now required set a new mark for dredging machines to reach, and most of the plant now in use is not only incapable of making these depths, but is more or less obsolete and defective. It is a rare thing to find a dredge well designed, well built, and well equipped, with even an approach to the standard which is found in modern shipbuilding and marine engineering. In many cases, the dredge is crudely designed with a miscellaneous collection of ill-shapen machinery crowded into inaccessible places, braced and strengthened with many afterthoughts; or it may be a high-class machine built at great cost with spotless decks and good intentions, but marred by errors in design. In either case the result may be the same, high operating cost, small output, and frequent delays and repairs.

The importance of being able to maintain a high average working time is well known by every dredge operator. Not only is a dredge earning nothing when it is standing still, but it delays a whole chain of plant depending on it and loses the money it should have earned, besides money expended on repairs. A certain amount of delay to dredging plant is unavoidable, such as that due to weather conditions in exposed locations, but the greater part of the delays that usually occur are preventable by having more perfect machinery under good management.

There are several factors that contribute to the ideal result of large output at minimum cost:

(1) Sound design in general scheme of adaptability to conditions, including a definite knowledge of weights, displacement, and trim to suit the conditions.
(2) Sound design in simplicity and excellence of detail, up to good marine engineering standards, especial attention being paid to economy of fuel and ease of operation with a small crew.
(3) Good workmanship by experienced builders.
(4) Co-ordination of all elements of the combined operation of dredging, fuel supply, disposal of material, and maintenance.

As to sound general design, it is essential that the plant should be perfectly adapted to the conditions under which it is to work. This remark would seem superfluous were it not that so many misfits are to be seen. This means, as a rule, a special machine suited to the special purpose. This is an age of specialization. In transportation, vessels are especially built for special trades. Railway cars are made for special purposes. Dredges should be designed to suit the work to be done not only with a knowledge of the mechanical detail, but of the engineering questions involved, and also with a knowledge of the results of operation and maintenance of similar plant. Only in this way can one know the difficulties with which one may have to contend and the defects to be remedied, and what to avoid as well as what to adopt. Many dredges are in use which show that the designer or builder did not know how deep in the water it would float until it was finished and tried, or whether it would be down by the head, or *vice versa*. The machinery and piping are frequently placed in awkward and inaccessible positions. Such errors as these are inexcusable.

There is a certain indefinable quality about good design that shows in the symmetry and balance of the general lines and in the arrangement and simplicity of detail.

There has been too much rule-of-thumb in dredge design and too much remedying of break-downs by increasing the size and weight of parts without regard to the question of whether the plan was good in the first place. It is always well to work with natural forces rather than against them. Nature will always rebel when her laws are transgressed, but is a good friend and ally when man works in harmony with her.

The value of efficient modern plant is well illustrated in the history of the Cape Cod Canal. In the early part of that work twenty-six dredges and shovels of antiquated type were used and made slow and costly progress. Later, two modern dredges were especially built for the purpose, which did about as much work as all the other plant put together. This result is described by William Barclay Parsons, M. Am. Soc. C. E., in his paper "The Cape Cod Canal"*, in which he states:

"It will be seen, therefore, that had specially designed dredges been set to work at the outset, they would have more than paid for themselves during the construction of the canal."

The truth of this statement hardly seems to require an argument, but the fact remains that owners and operators frequently hold on to their old plant with

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1.

great affection, and continue to pay large bills for repairs and inefficiency. The small contract system by which so much dredging work is done is responsible for the continuance of much small and inefficient plant. Where it is possible to carry out works of magnitude in continuity and allow proper time for preparation a much better class of plant can be secured.

The administration of port works in Europe and abroad is usually in the hands of a local commission or board of trustees for each place empowered to carry out the development of the port and maintain the works by a permanent staff. In this way an experienced and well equipped organization is built up which, as a rule, is more efficient than either contract work with its attendant uncertainty, or Federal Government work. Contract work is suitable for any job which is at an end when done, and for which the contractor is properly equipped, but the development of a growing port and the approaches thereto is a continuous operation and should have adequate plant especially designed for it.

The following is a brief description of the four principal types of dredge, with notes on modern design.

The Ladder Dredge.

This type of dredge is more largely used in Europe and abroad for general dredging operations, and the detail follows established practice. Only a few examples have been built in the United States. This type is suitable for making wide cuts in open spaces, as in approach channels, etc., where there is sea room for setting out the six anchorage lines required. It is also suitable for taking off a thin cut and leaving a smooth level bottom close to grade. A channel cut, 300 to 600 ft. wide, can be made at one time, and the buckets can fill on a cut 3 ft. in thickness.

This type of dredge has been open to the objection of high cost of maintenance, due to wear of the buckets and tumblers the renewal of which is expensive. This wear can now be minimized by better design, manganese steel pins and bushings, larger bearing surfaces, and renewable face plates for tumblers. For some years the writer has used a packed and lubricated joint for bucket pins and bushings, and grit-protected bearings, which have proved a great help. Main driving engines may be direct geared to the tumbler shaft instead of requiring the long transmission formerly used. Side anchorage lines can be carried on spuds submerged to a depth to clear tugs and scows so that there is no trouble from this source. Mooring winches are now frequently arranged for wire ropes carried to a central winch, and the entire dredge is planned for one-man control, so that the large crew formerly needed is reduced to a small number.

The steady speed of running of this type is suited to the economical use of steam expansively by modern marine methods, so that good economy of fuel can be had, and, under good conditions, more cubic yards of material can be raised with this type per ton of coal than any other. It is the only type in which the machinery load is balanced. In the dipper and grab, the work done in lifting the structural parts is lost and in the hydraulic dredge the transport of the water is a loss. The type is more seaworthy than the dipper or grab, as no spuds are used, and the ability to take light cuts over large areas and leave a level bottom close to grade renders it economical for channel deepening in the open or wherever there is sea

room to work and set out the anchorages. In European harbor works it is used for general purpose dredging, even in confined locations, and dock cleaning, etc. The writer believes that for such work the dipper or grab would be more suitable, being a self-contained machine standing on its spuds, with no moorings to obstruct navigation. On the other hand, engineers might profitably adopt a modified form of ladder dredge in the United States for conditions where it is especially suitable. Ladder dredges have been built in sizes up to 2-cu. yd. capacity of bucket, which, at 14 buckets per min., is equal to a capacity rate of 28 cu. yd. per min.

The Dipper Dredge.

To those who know the dipper dredge, no introduction is necessary. Built in a variety of forms and sizes, the type ranges from the light and small ¾-yd. machine, suitable for small drainage ditches, to the large and powerful harbor dredge with a 15-yd. dipper capable of working in hard material and deep water. Briefly, the dipper type is suited for general purpose dredging as in harbors, canals, and confined spaces where there is not room enough to set out the anchorages of a ladder dredge, and where disposal is by scows or casting on the bank. Having great concentrated power on a large bucket, it is better able to handle hard material, boulders, etc., than the ladder type which has less power distributed into a number of small buckets. The dipper dredge is more of a general purpose machine and can cover a wider range of conditions than the other types, which are more of a special nature and suited for their own class of work. It can be adapted to work in mud or soft material equally well, and for this purpose the spuds are fitted with bearing shoes of large area and the dipper is of larger size and without teeth.

No American dredge user would be without this indispensable machine, and as its merits become better known, it is finding its way into all the harbors of the world. As commonly used on the Great Lakes the dipper dredge has been limited to comparatively shallow depths and is generally of wooden construction, but for deep water and in order to conform to marine practice a different and better class of dredge is necessary.

Among the early dipper dredges of modern type were those designed by the writer in 1886 and thereafter for work on the Great Lakes, which established the type having a single direct wire-rope hoist, bowstring boom, and overhead **A**-frame. Previous to that time the prevailing type was fitted with a three-chain hoist and crane. These, in turn, have been superseded by later forms with improved detail, until there is now available a machine of the highest marine standard, which is capable of continuous heavy duty and which can be built for depths of 60 ft. and of large capacity and power.

Among the improvements which have been adopted in some of the writer's recent designs are the following:

1. Duplex parallel hoisting ropes, by which smaller ropes can be used over large sheaves for durability.
2. Solid laminated dipper arm, giving much greater strength and resiliency than the usual split arm, and eliminating rack and pinions.
3. Center crank engines for the larger sizes, conforming to marine practice and avoiding overhung cranks.
4. Power feed to the dipper arm as well as brakes, giving better control for manœuvring the dipper and for spotting over scow.

5. Improved type of boom with straight taper and with turn-table built solid with the boom, giving great strength and few parts; center of gravity close to axis of revolution, so that speed of movement is quickly attained.
6. Automatic backing line, in which the slack is taken up automatically and time saved in operating.
7. Improved type of band friction clutch, which does away with all sliding parts or slots in the shaft.
8. Air-operated controls throughout, doing away with mechanical levers.
9. Improved balanced elastic spud mounting for forward spuds by which concentrated stress is safely distributed and heavy loads are safely carried.
10. Improved method of reeving ropes for forward spuds, whereby all sheaves come above deck and the ropes are readily reeved and preserved from injury.

For improved fuel economy in the dipper dredge, much remains to be done. It has not been found advantageous to compound the cylinders of the main engines, as the work is too intermittent and quick responsive action to variable load is essential. For a two-crank engine, the cut-off must be late enough to give good starting power at all crank angles, which means about three-quarter stroke. Three or four-crank engines would give a more uniform torque and good starting power and permit earlier cut-off and consequent economy. Steam can be saved also by reducing the number of steam pipes frequently found on a dipper dredge to the lowest possible, and returning drainage to the boiler. The forward spuds may be worked from the main engines instead of from separate engines. The latter method is favored by some on the ground of saving of time in operation, but with the modern quick-acting friction clutch set by compressed air no loss of time occurs, and the condensation of additional engines is avoided. Such engines, being used intermittently, fill up with water between times.

Superheating and condensation will also contribute some benefit, and a three-crank engine for swinging will give a smoother motion and require less steam than a two-crank engine. Attention to all these points, and an ample and efficient boiler will do much to reduce the fuel bill.

The weak points of the prevailing type of dipper dredge are found in the spuds, dipper arm, and boom. As ordinarily used, these parts are a survival of the design first developed in the light and small dredges used on the Great Lakes and in the Middle West where they originated, and which are unsuitable for deep-water and marine service. The difference in conditions is quite comparable to that of a light river steamboat and an ocean-going vessel; no development of the river steamer by making it bigger and heavier would ever fit it for ocean service, and yet this is precisely what has been attempted in several examples of deep-water dredges evolved from lakes and river practice. For instance, one finds plain straight spuds working in plain straight slides of insufficient depth, a split dipper arm of great length and weight with racks and pinions, and a bowstring boom with its weak lower end incapable of resisting twisting.

The question of spuds for deep water is most important. For the early dredges in shallow water a plain straight stick of timber working in plain straight guides answered very well and still answers. The next step was steel spuds of box-girder form, but with these the failures have been many. They have not the elasticity of the timber spud, and fail by crushing, buckling, shearing rivets at the points of support, and also cause similar action on their supports. The con-

centrated load on the bottom of the hull is very severe. The middle part of the slide or casing contributes no support, and the load is concentrated at the point of contact, causing crushing of the fibers if of wood and shearing of the rivets if of steel.

The logical way to provide for these loads would be first to adopt a spud section that will give the required bending moment with the least weight, and then mount it in such a way that the load at the bottom support is distributed over a sufficient number of rivets to carry it within the limits of good bridge practice. The top support can be mounted in a cage containing multiple springs to obtain elasticity. The writer has developed a spud mounting* on this principle, in which the bottom bearing is formed by balanced shoes of large area, which accommodate themselves to the surging of the spud and in which the stress is received and distributed into the bottom plating of the hull. The spud casing is thus subject to no strain, being clear of the spud altogether, and merely serves to keep out the water. The elasticity provided by the springs at the top is of great value in easing sudden shocks and surging strains, and contributes greatly to the life of the structure. These springs may be M. C. B. standard car springs which are comparatively inexpensive and may be used in any number to provide for the load.

For moving these spuds up and down, wire ropes may be used, and, for large sizes, the ropes may be in multiple. In some designs, notably in the large Panama dredges, these ropes were reeved in a difficult and inaccessible manner, with the lower sheave so near the bottom that it got into the mud. In the writer's recent practice these ropes are arranged so that all the sheaves come above deck, and all the reeving and changing of the ropes can be done easily and quickly. Where required for special cases the dredge can be designed so as to be capable of shipping and unshiping its own spuds without the use of a derrick. This is a valuable feature in dredges that go to places remote from a repair yard, and is accomplished by designing the parts so that the spud can be slung up by the main hoist and shipped into place while the boom is swung to the side over the spud.

In dipper arms the usual construction is a split section in order to allow the hoisting rope to pass between. This, although satisfactory for small dredges at shallow depths, is quite unsuitable for heavy work in deep water. Increasing the size and weight, as has been done frequently, only aggravates the difficulty and puts a load on the pinion teeth and shroudings which is contrary to the most elementary principles of engineering. On this principle the dipper arms on the Panama dredges *Paraiso* and *Cascadas* are 12 by 22 in. in section, 72 ft. long, and weigh 40½ tons or, with the dipper, about 60 tons. This huge member is called on to dive and lift and twist and swing while riding on the racks and pinions and being confined in a saddle-block. According to reports, its life is only 6 months and that of a rebuilt arm 3 months, which is not surprising.†

For deep-water work and, indeed, for all work, the writer has discontinued the use of the split form and has developed a solid arm on the laminated principle, composed of wood and steel plates alternately. This combines great strength with resiliency, which is so necessary in working in hard material at great depths. This dipper arm will twist and spring to a considerable extent without injury or deforma-

* This mounting has been patented.

† *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 515.

tion. One of these dipper arms has been in service for six years in hard digging and is still practically as good as new. It is light in weight, strong and elastic, has large flat bearing surfaces and plenty of play with no rack and pinions to pinch and wear, and there is no appreciable strain against the saddle-block, as the pinion thrust is absent.

In boom design, the bowstring form is the early wooden boom reproduced in steel and increased in size. It is open to the inherent weakness of a curved lower member to resist the great lateral stress due to the dipper arm when swinging or when swinging is arrested. A long dipper arm constitutes a lever for twisting the boom and splitting it apart. Being confined at its heels such a boom is invariably weak near its lower end. Attempts to remedy the trouble have been made by greatly increasing the section and weight but without satisfactory result.

To meet this the writer developed some time ago a type of boom which has been used in many dredges and steam shovels with good results. It has a straight taper, is strongest near the lower end, and has wide flange members on bottom to take the lateral thrust of the dipper arm. The turn-table is solid with the boom and on deck instead of overhead. This type also possesses two further advantages of fundamental importance: First, it is mounted by its heels on the collar bearing on deck so that it can roll slightly, as well as be free vertically, and thus ease the stress due to the side thrust of the dipper arm. This flexibility is a great safeguard when working in deep water. The second advantage is improved visibility of all his work from the operator's stand on the upper deck. It is useful and convenient to have a bridge the whole width of the vessel on the spud framing, and the whole work can be commanded from this point.

These and other improved points of detail enable engineers now to deal with a greater range of work than before without sacrificing any essential advantage and without going to extreme lengths of members. For example, it is now possible to design a dipper dredge that can work in hard material to a depth of 60 ft., that will have a boom 65 ft. long, a digging angle in bottom position of 25°, and an A-frame 48 ft. high. These figures would be impossible to reach with the bowstring type of boom and split dipper arm, and they show that a depth which covers the requirements of modern deep-draft vessels, with margin for tide, can now be reached effectively.

The Hydraulic Dredge.

It is perhaps unnecessary to review at length the hydraulic dredge, the character and type of which are well understood. Briefly, the hydraulic dredge may be divided into four leading types:

1. The sea-going hopper type.
2. The lateral-feeding or ship-channel type, with floating discharge pipe.
3. The forward-feeding or Mississippi type.
4. The radial-feeding type, with spud anchorage.

These four types have been built in a great variety of forms to suit special conditions.

The hydraulic type is sensitive to conditions, and if they are unfavorable or the design not well adapted to them, the output falls off rapidly. On the other hand, for suitable conditions, and disposal considered, it is one of the cheapest methods

of dredging—hence the necessity of a careful study of conditions and of making the design to suit.

The principal source of delay and expense in this type is the renewal of worn parts. The wear of pumps and pipes is sometimes severe, especially in cases where high velocity is essential to good output. This wear varies with the kind of material. Thus, sharp sand mixed with stones and clay forms a particularly abrasive combination, while sand alone is less abrasive, and blue clay or mud not at all abrasive. To provide for wear on the interior of pumps, manganese steel linings are frequently used.

Points of recent improvement in hydraulic dredges are as follows:

1. Simpler design of pump with fewer parts, easier to take apart and to replace liners. Pump shells made in two pieces split horizontally. Easy bends and free passages. Adjustable wearing rings to inlet that can be tightened up without stopping pump.
2. Geared turbine drive, occupying less space and weight as compared with reciprocating engine, and permitting the use of a small fast-running pump.
3. More efficient cutter, especially for clay soils.
4. Better steam plant for fuel economy.
5. The use of all steel ball joints instead of rubber sleeves to connect floating pipes.
6. Elastic spring ball joints for pipe lines exposed to sea waves.

These points, and others which might be mentioned, combined with careful design, could not fail to result in marked economy. The geared turbine has now reached a stage which warrants its adoption for this class of work. Although some failures have occurred with reduction gears, both in marine and stationary work, they have been due to preventable causes. Accuracy of toothed surfaces, moderate loads, and perfect lubrication are essential. The great value of the turbine drive for this work is that it permits the design of a pump of small diameter and better proportion, and, therefore, it is cheaper and more efficient than a pump of large diameter made to suit the slow speed of reciprocating engines. As dredging pumps are from necessity side suction with overhung impeller, this reduction in the weight of the impeller is important as affecting the shaft and bearing.

In the earlier dredges and for free material almost any kind of cutter or agitator would suffice, but this type is now used for more tenacious materials such as clay and hardpan. This is due to the development of an efficient spiral-bladed cutter, of which the cutters designed by the writer for the dredge *J. Israel Tarte*, in 1901, and the *Francis T. Simmons*, in 1906, were pioneer examples. When properly proportioned and with the right clearance angles, these cutters excavate the toughest clay cleanly and feed it into the mouth of the suction pipe, so that the pumping part is really only the transporting apparatus to deliver the spoil.

The development of all-steel ball joints for pipe lines is a saving over the rubber sleeves formerly used and permits higher pressures. For moderate pressures a simple ball fitting into a parallel bore socket and self-packing with the grit of the material is sufficient. Such a joint is readily coupled and uncoupled. For heavy pressures or long-distance pumping a packed joint may be used. With elastic joints of strong pattern and fitted with spring buffers, dredges can now be worked in waves and exposed locations, which widens the range of work that can be done by this type.

For long-distance pumping it is generally preferable to use relay pumps, limiting each pump to a head of not more than 100 to 120 ft. Pumps can be driven to greater heads than this in single-stage, but the high speed which is no objection in a water pump, becomes a difficulty when working with a fluid that carries solid matter. Relay pumps can be suitably driven by electric motor for simplicity, and indeed the entire dredge can be so driven profitably where current is available.

The special feature of the hydraulic type of dredge when a pipe line is used is that it disposes of the material. This operation is costly in power as a large volume of water must be delivered to carry the spoil, and costly also in wear and tear, due to abrasion of pump and pipes. The net result, however, is quite economical as compared with other methods of disposal, and under conditions favorable to large output very low costs can be reached. The percentage of solid matter which can be carried may vary from 5 to 10 in heavy gravel or stones to 50 in soft mud, so that no definite figure can be named.

This system is well adapted to the reclamation of waste land, and in many cases the value of the reclaimed land will more than pay for the cost of the reclamation. There are enormous areas of marsh land adjoining the great cities that could be filled in by this method with economy and profit, and with powerful modern tools such as have been described, the cost would be reasonable.

With regard to the crab or clam-shell dredge, this type is well known and does not call for special description. It is exclusively adapted to soft material and finds a wide range of application outside the dredging field, as in handling coal, ore, and sand. As a dredge, it is specially adapted to such work as cleaning out slips, etc., in confined locations, but for hard material or work in the open, the other types are better.

In the foregoing brief remarks leading to the conclusions stated in the beginning, the writer has given a few specific points in which dredging machines are being improved, and has indicated the lines on which further improvement can be made. He has also touched on the larger question of the economics of shipping as being bound up with the general question of improvement of harbors, dredging plant, and marine transport, generally, and concerning which there is great need of the dissemination of further knowledge and discussion by competent men.

DISCUSSION

E. J. DENT,* M. AM. SOC. C. E. (by letter).—The author's statement, that the United States is behind other nations in respect to dredging machines and port equipment, should not be accepted without reserve. In the United States, the use of ladder dredges has not been approved to anything like the same extent as in foreign countries. In the matter of other forms of dredges, it is believed that the United States is well abreast, if not well in the lead, of foreign countries. On one occasion, the writer saw a harbor improvement made by means of ladder dredges and portable flumes for distributing the spoil over an area to be reclaimed. It was an impressive illustration of the use of a ladder dredge by a foreign engineer accustomed to such machines, where any American engineer would have unquestionably used a far more efficient pipe-line dredge.

In regard to the necessity for cargo vessels of 25 000-ton capacity, and even greater, the writer emphatically dissents from the statements made in the paper. He similarly dissents from statements to the effect that cargo vessels of the near future will require depths of 40 ft. or more.

While at sea, a vessel capable of carrying 25 000 tons of miscellaneous cargo could undoubtedly carry it more cheaply than the modern small cargo carrier. The cost, however, of assembling a miscellaneous cargo of that size in a position from which it can be rapidly and efficiently transferred to a ship's hold, the cost of transferring such a cargo to the ship's hold, the cost of removing the cargo from the ship's hold at the port of destination, and the cost of breaking up the cargo into small units for distribution at the port of destination, must all be considered in connection with the total cost. A small cargo carrier has many advantages over a large carrier under present and reasonably prospective conditions.

In 1914, there were only thirteen ships afloat, drawing more than 33 ft. of water, that had entered or left New York Harbor in the preceding two years. With only two exceptions (the *President Grant* and the *President Lincoln*), these thirteen ships were trans-Atlantic passenger and express vessels. The amount of cargo handled by them was relatively small. The two exceptions were generally understood to have been commercial failures. The maximum draft of any of the thirteen vessels was, as the writer now recalls, about 39 ft. for the *Imperator*.

A recent compilation taken from the register of ships owned by the United States Shipping Board, in January, 1920, showed the maximum summer draft of these ships, exclusive of former German ships, to be as follows:

Draft.	Number of ships.
20 ft. to 20 ft. 11 in.	24
21 " " 21 " 11 "	116
22 " " 22 " 11 "	141
23 " " 23 " 11 "	302
24 " " 24 " 11 "	557
25 " " 25 " 11 "	137
26 " " 26 " 11 "	106
27 " " 27 " 11 "	56
28 " " 28 " 11 "	24
More than 29 ft.	8

* Lt-Col., Corps of Engrs., U. S. A., New Orleans, La.

The smaller boats built on the Great Lakes, with horizontal dimensions fixed by the Wellington Canal, may be too small for general ocean traffic, but no such criticism is applicable to the larger standard freight boats owned by the United States Shipping Board. The following dimensions of two such boats are instructive:

	Westlake.	*Waban.*
Dead weight, in gross tons, summer	8 548	9 428
Length	410 ft. 5½ in.	402 ft. 6 in.
Breadth	54 ft.	53 ft.
Draft	24 ft. 1½ in.	26 ft. 6¾ in.
Daily fuel consumption, in tons	42	52½
Speed, in knots	10½	11

There are relatively few harbors in the world where vessels drawing as much as 35 ft. can now enter. There are very few vessels in the world which require a greater depth than 35 ft., and such vessels are engaged practically exclusively in the North Atlantic passenger business between New York City and a few selected ports in Europe.

In regard to hydraulic dredges, the writer feels that Mr. Robinson's paper is timely, in that it calls attention to deficiencies in the way such plants are designed at the present time. The "rule-of-thumb" method in designing and in remedying defects that develop, has been followed to such an extent as seriously to discourage the engineer who wishes to work on a more scientific basis. Opportunities for improvement are everywhere present, but the "practical man" is nearly always in the saddle.

Slow-speed, reciprocating engines, direct-connected to dredge pumps of large diameter to make up for deficiency in speed, have become the rule throughout the United States. It is not generally recognized that large diameters inevitably result in low efficiencies, as well as in unwieldly and costly parts. In electrically driven dredges, suitable pumps of small diameter are already used, and if suitable driving mechanism was available, the smaller pumps would soon come into general use. The reduction gear should be tried out and perfected for this class of work.

Many, if not most, of the existing pipe-line dredges have their capacity limited by the capacity of the cutter to loosen the material whenever they encounter anything except the softest kind of harbor mud. Whenever a large project is undertaken, involving the excavation of tough clays or other material which can be loosened only with difficulty, provisions for a special plant should be considered, and, in many cases, the price of a special plant will be paid back before the job is finished. The cost of operating an hydraulic dredge is affected to a very small degree by the yardage per hour, and the capacity of the pump and pipe line is, in tough materials, ordinarily much greater than the capacity of the cutter to loosen the material.

In pipe-line dredge work, where the material is to be transported long distances, the use of pipe lines larger than 20 in. in diameter is frequently desirable. The maintenance of a suitable pipe-line velocity through more than 4 000 ft. of pipe 20 in. in diameter requires an excessive pressure at the stern of the dredge,

but for shorter lengths of pipe a 20-in. line can frequently carry all the yardage the cutter is able to loosen. When long pipe lines are to be used, the problems involved in the use of a 20-in. line and relay pumps, and the use of pipe of larger diameter and a single pumping plant, should be figured for the specific case.

WALTER S. WHEELER,* M. AM. SOC. C. E. (by letter).—Too much emphasis cannot be placed on the need of better dredges. Probably as much time is devoted nowadays to the repair of dredges as to the actual dredging.

As to the need of channels of a depth greater than 40 or 45 ft., the writer is not so certain. A channel dredged to a greater depth, as far as the extra depth is concerned, would be earning nothing, as well as the dredge built especially for digging this extra depth, for many years to come, and such a dredge would be obsolete by the time such depths could be utilized. It would be used during its working life for digging shallower channels, and that would be as uneconomical as the larger ships not loaded to their full capacity.

In order to insure a working depth of 40 ft., the entrances to the Panama Canal have been dredged to a depth of 42 ft., by ladder and dipper dredges in rock and stiff clay, and by suction dredges in soft material. The writer has known of places where the sea-going suction dredge, *Caribbean*, has excavated to depths of more than 50 ft. at the Atlantic entrance to the Canal.

The suction dredges on maintenance can easily take care of the silting, and a good drill barge and ladder dredge could easily take out another 3 ft. or more of rock, if it becomes necessary.

In the case of the Panama Canal, the width of the locks would probably be a limiting feature rather than the depth, as they can only accommodate ships of something less than 110-ft. beam. Rebuilding the locks would be a much greater item of cost than digging the Canal deeper, which could easily be done by the equipment in use to-day, provided a depth of not more than 45 ft. was desired.

However, anything that the writer has said should not detract from the fact that better dredges are needed, both in design and equipment, and it would certainly seem that some of the many expert ship designers could profitably give valuable aid to the less expert dredge designers.

GEORGE B. MASSEY,† M. AM. SOC. C. E. (by letter).—It may not be realized by the average engineer in the East that most of the dredging machinery used in the United States is built in the district which has Chicago as a center; and it is a source of the greatest satisfaction to engineers of this district, deeply interested as they are in the proposed international waterway from the Great Lakes to the sea, to know that Canadian engineers and Canadian dredge builders stand ready to do their part in its accomplishment. A large part of the work on this waterway is in Canada, and our neighbor on the north is to be congratulated that she is able to handle her part of this immense programme without assistance.

The writer spoke of Chicago as the center of dredge building for the United States. In South Milwaukee, Wis., is the plant of the Bucyrus Company, the builders of several hundred dredges which are operating all over the world. This Company built the three largest Panama dredges which have been so successful in handling the slides in the Panama Canal after the Scotch ladder dredge had proven

* Dover, N. H.
† Chicago, Ill.

a failure for that work. In Marion, Ohio, are the works of the Marion Steam Shovel and Dredge Company which probably has built more dipper dredges of moderate size than any other company in the world. In Marion, also, is the plant of the Osgood Company, which has built a great many dredges for the Atlantic seaboard.

Mr. Robinson's reference to the Cape Cod Canal is interesting, as the writer called on William Barclay Parsons, M. Am. Soc. C. E., when the plans for the Canal were in the early stages. The question of new plant especially designed for the work was considered, and when the large outlay required was brought into the discussion, Mr. Parsons stated that he thought he could persuade the Canal Company that modern, powerful machinery was necessary to obtain results on the work, even if its first cost was considerable. It was in the nature of a surprise, therefore, when the old dredges assembled on the work. On projects of this magnitude, even heavier duty dredges than those finally purchased would have been good policy, even at the additional cost.

Many of Mr. Robinson's remarks on the ladder dredge are well taken, but in the light of the experience of the past fifteen or twenty years with placer dredges, a little additional evidence as to the value of lubricated joints for bucket pins and bushings would be in order. Such joints were illustrated in *Die Baumaschinen*, published at Leipzig, in 1897. The Bucyrus Company states that such pins and bushings were introduced on its early placer dredges by Mr. Robinson, when he was Chief Engineer of that Company. By 1900, such devices had been found to be ineffective, and the practice of using high-grade steels with the necessary wear-resisting properties was inaugurated and has been further developed up to the present time.

On its dipper dredges, the Marion Company has used twin hoisting ropes successfully for years. The use of center-crank engines is not so important, as side-crank engines properly designed have outworn other parts of dredges and, after working for years, have been transferred to new dredges.

The Bucyrus Company has applied power feed to the dipper arm, but it is the exception rather than the rule. The straight taper boom with turn-table built solid was in use on two dredges on the Panama Canal for several years before the large Bucyrus dredges with bowstring booms were purchased. The standard band friction clutch of the Marion Company does away with all sliding parts and slots in the shaft. Air or steam-operated controls throughout are standard on all modern large dredges built in the United States to-day.

The old Osgood dredges were always built with the overhead gallows frame for hoisting the spuds, and the third large Panama dredge built by the Bucyrus Company is so equipped. This design is the logical one.

Compound engines have been used only in a few cases on dipper dredges. The Bucyrus Company states that the first case in its practice was a 6 or 8-yd. dipper dredge built for George H. Breyman and Brothers. The Company found the operation to be satisfactory and later built two other dredges with the same kind of engines. From the record of the latest Breyman dredge, the Chief Engineer of the Isthmian Canal Commission decided to continue the use of compound engines on the 15-yd. Panama dredges. The compound engines show a sufficient saving in water and coal to justify the extra parts when at work in outer harbors, etc.,

where the transportation of water and coal is an item. As far as their performance in digging is concerned, it has been entirely satisfactory.

The three or four-crank engine does not justify its cost, in the writer's opinion, when two-cylinder simple engines with link-valve gear and piston valves can be operated with the linking-up lever instead of the throttle.

The use of the main engines for handling the spuds is almost universal practice on small dipper dredges. There is no objection to their use for this purpose on large dredges, provided the backing and swinging are done by other engines. The spring spud mounting has been used for the spuds of placer dredges for years. The writer has a photograph which he took in California in 1910 showing M. C. B. car springs used on the upper spud bearing. The shipping and unshipping of the forward spuds by the boom has not been done in the United States, so far as the writer knows.

The single dipper stick would seem to be a step in the right direction. It is possible, of course, only with twin hoisting ropes.* This type of handle has been used for years by the Bay City Dredge Company on its dry land dredges, and is of circular cross-section, built very light, and without pinions or racks. The construction of these handles enables the operator to lower the dipper on the sloping side of the spoil bank or dredged material on a scow and roll out any large rock which will not pass through the dipper.

The Bucyrus Company goes to the other extreme on its 300-ton shovels, the boom not being split for the handle, the latter consisting of two members, one on each side of the boom. The contention on the part of the Bucyrus Company is that the swinging circle should exert its effort to swing the boom at a point as near as practicable to the center of gravity of the boom and other swinging parts. This locates the swinging circle above the main deck, leaving free space for moving about and handling lines.

Mr. Robinson omits mention of the most recent improvement in centrifugal pump design for dredging purposes, namely, the increase of clearance passages through the pump for stones, sticks, and trash, by cutting down the number of blades and changing their shape.†

The clam-shell dredge has been developed to its greatest perfection in California where dredges are in use carrying booms more than 200 ft. long and buckets up to 4½ cu. yd. These machines are not pinned up on spuds, but roll as the boom swings. They are not subjected to digging strains and do not require the strength of hull that a dipper dredge does. The machinery is simple, few men are required and where the long reach is desirable or where the work is easy or at great depth, they have an advantage over the dipper dredge.

WALTER FERRIS,‡ ESQ. (by letter).—The writer has read with much interest Mr. Robinson's comments on the design of dredges suitable to meet the requirements for deeper harbors which he outlines in his paper. Although any engineer familiar with the development and use of dredges will agree with many of his

* Discussion of the paper by Ray W. Berdeau, Assoc. M. Am. Soc. C. E., on "The Three 15-Cubic Yard Dipper-Dredges, *Gamboa*, *Paraiso*, and *Cascadas*, as Supplied and Used on the Panama Canal," by William M. Rosewater, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 538.

† "A Dredging Pump of Novel Construction", by W. J. White, *Journal*, Am. Soc. of Mech. Engrs., January, 1920.

‡ Cons. Engr., Bucyrus Co., South Milwaukee, Wis.

comments, there are some points on which a supplementary statement might help to give a more accurate impression.

No objection can be made to Mr. Robinson's statement of four factors which contribute to ideal dredge performance, which may be briefly summarized as: (1) sound general design; (2) sound detail design; (3) good workmanship; and, (4) co-ordination. The latter may be said to be included in Factors (1) and (2).

There is, however, another factor which influences very largely the design of dredges built by concerns of reputation and standing for customers who are experienced in the use of the class of equipment purchased. This element is conservatism, including especially a subordination of all tendency to introduce attractive new designs to the necessity for insuring successful operation. Purchasers of dredge plants may be roughly classified as experts or as novices in the use of excavating machinery. The expert is apt to be a successful contractor who knows that continuous operation is far more important than untried improvements. Although this purchaser wishes the best possible machine, he gives great weight to excluding any possibility of failure from an unforeseen cause, and desires that the design of his machine shall be carried out along tested lines. The other, or inexpert customer, such as a harbor board just embarking on improvements, is more open to suggestions from his dredge builder and looks more favorably on changes from established practice.

The result is that the engineers of the largest and most reliable concerns are apt to produce very conservative designs which at first sight might seem to be due to lack of inventive ability, but which are really attributable to close co-ordination between the designer of the dredge and an experienced user. The general habit of conservatism developed in such a dredge-building concern rests primarily on the imperative demand of purchasers for reliability rather than novelty.

A good illustration of these conditions is found in the history of the three Panama dredges. The writer was intimately concerned in the design and construction of these dredges from their inception, and a number of improvements were suggested to the Chief Engineer of the Canal Commission before the first dredges were built. Among these improvements was the use of duplex parallel hoisting ropes, as suggested by Mr. Robinson. The change was not adopted, as there was some discussion as to the feasibility of successfully equalizing the strains on the two ropes and the performance of the single rope could be quite accurately forecast. Delay in delivery was also an important consideration.

For similar reasons a suggestion to omit the hull trusses which were developed in connection with wooden hull design and could theoretically be replaced in a steel hull by between-deck bulkheads, was rejected. The first Panama dredges were based on the known performance of the 8-yd. dipper dredges previously built by the Bucyrus Company, and the Chief Engineer of the Canal Commission wished to introduce no changes which might possibly prove to impair the strength or efficiency of the dredges.

Many additional points might be cited in the design of these and of other dredges to show that the omissions of the latest improvements is dictated by other reasons than lack of progressiveness or inventive ability. To show the different attitude which may reasonably be taken on the same proposed change, the writer will run through Mr. Robinson's list of ten improvements which he has adopted

in his recent designs and comment on the practice of the Bucyrus Company with reference to them:

1.—The duplex parallel hoisting ropes have been introduced by the Bucyrus Company on two or three different machines. Where the size is small, it is doubtful whether they result in improved service. In the case of the largest dipper dredges, the Company believes that the maximum economical size for single ropes has been exceeded and twin ropes should be used.

2.—The solid laminated dipper arm is an interesting patented device. The Bucyrus Company has no record of its performance, except that given by Mr. Robinson, and the writer is not in a position to comment thereon.

3.—The Company can see no reason for using center-crank engines, involving either expensive forgings or the possibility of defective steel castings, except when it is absolutely necessary to get a coupling or driving pinion on the outer end of the crank shaft. So far as is known, the only reason for the center-crank engine in marine practice is the necessity of coupling the crank shaft to the propeller shaft, and overhung cranks are generally used in all engine practice where they will answer the purpose.

4.—Power feed to the dipper arm: This consists of mounting a separate engine on the boom as is done in steam-shovel practice. This Company has built about ninety dipper dredges and the writer only recalls two of these on which the boom engine, with its attendant increase in maintenance cost and steam consumption, has been installed.

5.—Straight tapered boom built solid with turn-table: This is a patented improvement which was introduced on a considerable number of steam shovels and dredges. The Bucyrus Company has had extensive experience at maintaining steam shovels, and it is undoubtedly a fact that this boom as built does not stand up to the same hard usage as the ordinary types of boom. Apparently, there is no inherent reason for this, but the actual record does not sustain the claim of advantage.

In regard to its use on dredges, it is to be noted that two dredges with this patented boom were in use on the Panama Canal before the Bucyrus Company was asked to submit a design for the first two of its 15-yd. dredges (*Gamboa* and *Paraiso*), and no suggestion was made to the Company of the desirability of the straight tapered boom, built solidly with the swing circle. This, of course, may be due to the fact that the existing booms of this type had been built too light and had not made the favorable impression that their merits may warrant.

6.—Automatic backing line: In this item, Mr. Robinson doubtless refers to some particular design of automatic backing line, as the writer believes that the first device for backing up the slack automatically was built by one of the earliest Canadian engineers, Mr. Canan. In recent dredge practice, wherever the backing drum has been geared from the main drum, he believes it is customary to let the backing clutch drag slightly so as to take up this slack automatically. On a 6-yd. dipper dredge built by the Bucyrus Company in 1905, a counterweighted backing drum was furnished, driven by an independent backing engine, so that when the friction clutch was disengaged the counterweight would automatically take up the slack of the backing rope. This, like many other similar devices, although

mechanically entirely successful, did not show sufficient importance to create a continued demand.

7.—Improved type of band friction clutch: There are a number of band friction clutches in use which answer to the description given and avoid sliding parts or slots in the shaft, and the writer believes that the Bucyrus Company would receive more protests than commendations from its customers should it discontinue the use of its standard clutches which have been proved by thirty years of use, for the sake of avoiding slotted shafts. The conservative suggestion is to make the shaft big enough and retain the old design.

8.—Air-operated controls: Properly designed, such controls should be an improvement, the principal value perhaps residing in saving the draftsman's time in careful layout of operating levers and the erector's time in installing them. The Bucyrus Company has equipped a few dredges in this way with generally satisfactory results. When the customer is consulted in regard to them, he usually prefers the direct mechanical connection, believing it to be more reliable.

9.—Improved spud mounting: The upper ends of placer-dredge spuds have been carried on springs for many years past. Dipper-dredge spuds of the same construction may possibly be economical, but as the direction of force varies greatly, the detail design is more difficult and more likely to be defective.

For resisting strains from the spud against the bottom of the hull, it is believed that sufficient reinforcement can be incorporated in the hull without difficulty, and the writer doubts whether the special rockers are an improvement at this point in a dipper dredge. Of course, the length of the spud casing should be ample. If springs are used at the top of the spud casing, permitting larger angular movements of the spud, the rockers are probably necessary at the bottom; at least, this seems to be the case in placer dredges.

Experience in the matter of devices for steam economy again illustrates the divergences between what would appear to be good steam-using practice and actual demands of customers. Although there are some indications of a greater willingness to take proper care of a highly developed steam plant, the general attitude still is that a few pounds of steam are of very small importance compared with a yard of material excavated.

In commenting on the Panama dredges, Mr. Robinson criticizes the spud-rope reeving as difficult and inaccessible, with the lower sheave too near the bottom. The criticism is quite justified, but he omits to mention that in the third dredge the spuds were suspended from an overhead gantry and all the sheaves were above the deck line. The design in the first two dredges resulted merely from a continuation of past practice with a number of parts of rope increased from two to four.

Referring to the mounting of booms and swing circles in large dipper dredges, the Company is inclined to adhere to the practice of placing the swing circle on the upper deck, with the jaws engaging the boom approximately in the plane of its center of gravity. When the loaded dipper is being swung around to the scow it stands approximately at the height of the swing circle and there is little tendency to trip the boom. This construction also permits the boom to rock in its sockets on occasion without tilting the swing circle, giving, perhaps, greater flexibility in this regard than the construction advocated by the author.

Referring to the author's comments on hydraulic dredges, it is suggested that a dredging pump can be made more accessible when designed so as to remove the runner, with a short spud shaft through the front head opening, than when split horizontally. The recent practice of the Bucyrus Company is to omit casing liners entirely, lining only the front and back heads. The casing is then made of high grade steel with the thickness varying according to zones of wear.

The author's point regarding the geared turbine drive for drums seems to be well taken, subject to investigation of the behavior of the turbine when running at considerably below normal speed.

CHARLES EVAN FOWLER,* M. AM. SOC. C. E. (by letter).—The author of this paper seems to be unaware of the vast difference between the great passenger ships, or liners, and the great number of modest cargo carriers, comprising about 90% of the total tonnage in operation. The latter are, of necessity, of light draft and low power, built to run in shallow harbors at low operating costs. The vast supply of coal required for a great ship of 700 to 1 000 ft. in length limits its use and the number of ports to which it can run. The ports that will be able to provide even 40-ft. depths will for many years be not more than two or three on the Atlantic Coast of North America, and there is probably no more than that number for both Great Britain and Europe.

The hundreds of millions necessary for creating such harbors throughout the world precludes their possibility. The tens of millions required for giving increased depths for New York Harbor alone makes such an improvement of doubtful attainment for the near future. There is, moreover, a likelihood that much cheap cargo will again be carried by sailing vessels or auxiliary schooners; and had the Emergency Fleet officials listened to experienced operators and builders of the West Coast, there would have been a large fleet of such useful wooden vessels now in commission, instead of the present great idle fleet of wooden "jokesters." The writer found even the old Maine sailing barks perfectly satisfactory for sending millions of feet of lumber for Manila harbor work, one of them making a crossing in 35 days, and all of them delivering cargo so as to cause no delay on that $4 000 000 harbor improvement.

What is needed is better harbor arrangement and more cargo-handling machinery. The majority of harbors simply grow by accretion, with no general scheme of development, until at last there is a great aggregation of piers and slips as at New York Harbor, with no reason for there being as they are nor where they are. The planning of new harbors and the replanning of old ones must go hand in hand with the determination as to the future motive power for ships, and as to where it is to come from. Coal and oil for fuel are even now questions of serious concern, and it cannot be expected that hydro-electric power will be used to any great extent for anything beyond land purposes. Water power is always available on land, as fast as developments are made; and always power from winds at sea may be had merely for the taking. True conservation requires the fullest possible use of both natural sources of power.

The writer takes exception to the author's statements about the work of the United States Corps of Engineers, and the work of dredging and harbor contractors in this country. History shows there is no parallel anywhere to

* New York City.

the record and work of the U. S. Engineers both as to character and efficiency, and the troubles in harbor development so often ascribed to them, really come from our "pork barrel" river and harbor legislation. Very seldom is sufficient money appropriated to complete an entire project, and work done is often lost or obliterated before another appropriation can be obtained. Much of the inefficiency ascribed to the harbor contractor results from the same cause. It would be found to be a calamity indeed if the control and improvement of Federal waterways were taken from the Corps, and placed under scores and perhaps hundreds of authorities.

The ladder dredge has been found to have no place in our harbor work, due to the fact that other types of dredges have been developed, any one of which will for a given case produce better results. Its use therefore is confined to mining, or, in a modified form, to commercial sand and gravel digging.

The dipper dredge is purely a tool of practical development, and the application of rigid scientific analysis and design is not likely ever to produce a dredge that will not require extensive alterations on the actual work for which its use was planned, and still further changes and modifications when moved to another class of digging. The office design of a 7-yd. dipper machine produced many parts that had to be rebuilt when the writer tried it out on the work, and it finally became an efficient 3½ to 4-cu. yd. dipper machine. The Panama dredges supposed to be the last word in dipper-dredge design, have been, from all accounts, subjected to an extensive course of changes and modifications. There is not much doubt but that the new spud scheme, the new type of dipper handle, and the new this and that, as designed by the author, will be scarcely recognizable after the dredge captain tries them out on the work, and lucky indeed will be the scheme that lasts out the vicissitudes of five years of operation.

The suction dredge, as developed practically, is so much superior to any machine as yet to be built from theoretical plans, that one knowing both sides is loathe to discard so quickly many things of practical good as does the author. The fact is, that the good things developed on the work have been patented or kept secret to such an extent by the dredging contractors that it is no unusual thing for some one to discover "anew" wonderful dredge details, as shown in recent numbers of the engineering journals, that have been known for a long time to practical dredge men, and in some cases discarded as not worth while. The writer has a great many data, gathered while in charge for several of the larger concerns, and knows whereof he speaks. There is no mystery, as the author states, in regard to the quantity of material of any kind that can be carried in any size or length of pipe line. Can the author point to any case where greater efficiency has been obtained, than in the pumping by a "practical" electric 22-in. dredge with a capacity of 37 000 cu. yd. of material in 24 hours, a distance of 4 000 feet? Has the record of 21 hours running time per day for month after month been exceeded? Why, if rubber sleeves are so poor, are they still used on pontoon lines almost exclusively by the contractors who must make money? There are, on the other hand, a few places where ball joints are satisfactory.

There is no brief to be presented for a contractor who uses an inefficient or obsolete type of dredge, nor yet for the engineer who allows him to do so. The character of plant must be right, and the kind of machine the proper one. There

are many places where a clam-shell dredge can be used economically on large work and on fairly hard digging, or on rock pick-up work. There are also places where a suction dredge can be used in hard digging by adopting a clay cutter such as was developed before 1900, or even for digging some kinds of rock with a rock cutter, and pumping it ashore. Let us be fair enough to give credit to a great number of men who, in past years and in a practical way, have developed dredges to their present state of efficiency. Let us not blame them without cause for holding back facts and data which are still their stock in trade.

J. VIPOND DAVIES,* M. AM. SOC. C. E.—The author has presented a valuable paper on a subject which at this post-war period is of intense interest; and by no means the least element of interest in the paper is the suggestion it brings squarely before us as to the economics of water-borne transportation. The subject is of world-wide importance, as larger ships and larger ports would be of little value to the United States if other countries were not working in the same direction and at the same time.

Ports are an essential element in the commerce of any country where the railroad and inland waterway transportation meets the ocean-borne transportation, and thus obviously what concerns the welfare and development of the ports, as a means of conducting transportation, affects not only the port itself, but the entire country and nation.

In looking at this subject, the first factor for consideration appears to the speaker to be the ship. What are its needs and requirements? As the author suggests, the commerce of the world is divided largely into trade routes. There are, first, the great trade routes involving a comparatively small number of ports all over the world, in which service a large size vessel is requisite and desirable. Then, there are secondary trade routes, in which a very much larger number of ports become involved, and usually a smaller class of vessel is utilized; and then, supplementary to these, are the very large mass of port facilities used in the operation of local distribution within limited distances. Most of the great shipping lines have vessels which serve all these separate functions, adapting the service rendered to the volume of business and the character of the port facilities which exist and can be utilized.

In the necessities for port development is involved also the segregation of passenger service from freight service. In this matter the Atlantic ferry is like nothing else in the world. On this ferry between New York City and the near-by European ports, a class of ship is operated, which would be useless in any other service or on any other route. These vessels, of which the *Aquitania* and *Olympic* are the leading representatives, operate high-class passenger service with practically no freight service whatever. These ships necessitate special treatment of port development which is not applicable to any other classes, and are of sizes suited to the particular business they carry, but unlike anything operated elsewhere or in any other trade. Consequently, this particular class of vessels has necessitated special port development to meet the urgency required in speed of travel and extravagant luxury on this ferry route.

In considering the question of the ships in use as necessary for the study of port requirements, it occurred to the speaker to take the records of "Lloyd's

* New York City.

Register", using the 1917 edition, in order to get some information as to the character of vessels operating on various trade routes. The records of that date are undoubtedly incorrect now, due to the war losses as well as to the recent building deliveries, but the relative figures give fairly well the information sought. It may be noted, in the first place, that a vessel of 10 000 gross register tonnage will approximate 500 ft. in length and will draw, when loaded, approximately 27 ft. in salt water. A vessel of 15 000 gross register tonnage will have a length of nearly 600 ft. and a loaded draft in salt water of approximately 32 to 33 ft. Up to the limit of this size of vessel, ship construction can be economically designed with the usual port channel depths available. It is beyond this limit that economic loss occurs.

For the purpose of drawing comparisons the speaker has included the vessels of the great German lines as they existed prior to the war period, but which since that time have been absorbed by various nations. He has taken into account only the regular lines, as entirely distinct from what are known as tramp steamers (the ordinary tramp steamer doing a purely freight traffic), most of which are of smaller size than the regular liners.

Table 1 summarizes the information obtained and shows the allocation of the vessels of these various lines to the several trades. The largest vessel now used in the South American trade is about 16 000 gross register tons, and the largest engaged in the South African trade is of 13 361 gross register tonnage.

TABLE 1.

Trade route service.	Total vessels used in service by regular lines.	Number of vessels over 10 000 tons gross register.	Number of vessels over 15 000 tons gross register.
North Atlantic	553	118	33
South American	151	17	5
Eastern seas and Australian	507	23	None
South African	192	10	None

Not a single vessel to-day of 20 000 tons burden is in use on any seas except the North Atlantic. Since, as the author points out, the economics of operation point clearly to the necessity for big ships for long voyages, operated at a strictly economical speed and on the assumption that full freight cargoes are obtainable for the particular trade routes, then there can be only one conclusion drawn, that nowhere, except on either side of the North Atlantic, do the channels, canals, and ports permit of the use of any vessels larger than about 16 000 tons. Obviously, there is work ahead for engineers.

From these figures it will be seen that in respect to the size of vessels and port facilities, the North Atlantic trade is in a class entirely by itself, and is hardly comparable with any other traffic on the main trade routes; consequently, the ports on either side of the Atlantic which serve this North Atlantic trade must of necessity be considered by themselves as differing from all other port developments, and, in respect of this service, the St. Lawrence River, Boston, and New York on the western shores, with the Clyde, Liverpool, Southampton, and London, and the continental ports of Havre, Cherbourg, Rotterdam, Hamburg, and

Bremen, are the only ports which at present serve the business and necessitate special consideration. The growth of ship sizes in the past 50 years has been quite phenomenal, and there is no doubt but that the trend of increasing sizes in the next 50 years will indicate fully as great a growth as that in the past. The reference given by the author to the economy of the larger ship is noteworthy, and emphasizes the unfortunate policy possibly necessary for and carried out during the war period, of building in the United States such a large tonnage of vessels from 3 500 to 5 000 tons, for which the only future can possibly be a coastwise or local service for which these vessels are already nearly valueless as ocean-going craft.

The ship designer has done wonders in building the great passenger liners serving New York within the limitations of depth which have been forced on him and which undoubtedly involve most uneconomical construction, notwithstanding the fact that by lifting the structural main deck, and consequent effective depth of the vessel as a girder, to a point very much above water level, he has been able to build a safe ship, but at great expense. The largest vessels now afloat do not exceed 900 ft., but one can see in the immediate future vessels of 1 000 ft., and if engineers take a lesson by the past this length will assuredly be materially exceeded, and port provision must be made for these enormous vessels, not only in the North Atlantic ports, but, to a less degree, for similar steady and continuous increase in sizes in all other ports of the world, as well as in such important waterways as the Suez and Panama Canals. From the tendency in the growth of ships, it is clear that with the larger units and the enormously increased capital cost of each unit, the utmost effort must be made to reduce the charges running on vessels tied up in port and to get the greatest service out of such vessels by reducing the time of loading and unloading at port terminals. With ship-building prices as they are to-day—for high-class freight vessels approximately $180 per ton in the United States, or $150 per ton in Great Britain—the per diem value of a large vessel is a very serious item for consideration.

The contemplation of this aspect of the case extends the subject beyond simply the larger units and deeper channels into the question of general port development, involving the great terminal problems, including the handling of freight and the bunkering of fuel. Several of the largest and most costly vessels have recently changed from coal to liquid fuel largely for the purpose of reducing the difficulties of port delays. As tending to the increased size of units, the general trend of cost of labor and fuel, not only in the United States, but in all countries of the world, is a largely influencing factor, reducing, on the basis of tonnage carried, the amount of labor as well as the relative quantity of fuel, not only decreasing cost of operation, but lessening difficulties in carrying on the service.

To provide for future increase in sizes of vessels, in no port should there be any narrow restrictions placed on the navigable waterways, which might interfere with the normal development of the port. There has for years past been a tendency in the Port of New York (and this tendency is not confined to New York) to demand extensions of pier-head lines and thereby narrow the free waterways. In this country, the U. S. Corps of Engineers has steadily endeavored to prevent

extensions of pier-head lines, but frequently has been overruled by political influences. Such extensions granted may be very short-sighted policy which in the future is likely to react seriously on the possibilities of port development. With the enormously increased sizes of ship units, it should further be carefully guarded that no bridges be built over navigable waterways, which do not provide amply adequate head-room above water level, and under no conditions should piers be permitted in rivers or navigable waterways in which ocean traffic, now or in the future, may be operated. Similarly, in the event of the construction of tunnels below waterways, it should be carefully guarded that the upper limits of construction should not be placed at any elevation which would preclude a clear depth of at least 50 ft. of water in the waterways.

In respect to port development generally, the main difficulty in maintaining adequate channels is occasioned by the outgoing loaded vessel and not by the incoming vessel which has burned its bunkers during the voyage and therefore floats light. Unfortunately for this situation many of our important ports, such as those on the St. Lawrence River, New Orleans, on the Mississippi River, Philadelphia, on the Delaware, and the Hudson River section of the Port of New York, have their vessels loaded in either fresh or brackish water and, consequently, should be able to load to a deeper draft than they will carry when they reach salt water; consequently, the channels through which the vessel has to pass before it reaches salt water are the limiting factor.

The present situation in New York is of material interest. The main channels have been dredged for a depth of 40 ft. The big freight liners have been built for, and should be loaded to, a depth of approximately 37 to 38 ft.; as a matter of fact, they are unable to load below 35 ft., partly on account of the difficulty of maintaining the 40-ft. channel, but also because of the inadequacy of the waterway in the Hudson River above the Upper Bay. A 2 000-ft. channel is now being deepened by dredging by the United States Government by skimming over the bed of the river, removing a few inches of silt at a time (the method successfully used in the Ambrose Channel), in order to obtain within the Hudson River equal water to the channels through the Bay and out to sea. At the present time, it does not appear that the full 40-ft. channel is maintained throughout, so that the uneconomical use of steamships in the Port of New York is, as outlined by the author, materially affected by this insufficiency.

Most of our ports were originally developed and located in the early days of sailing vessels which desired to get well up inland for shelter and protection, and this has involved many great ports being at great distances from the open sea with all the disadvantages of river navigation and with a future of continuous and most costly dredging to maintain proper waterways. This seems to point very clearly to the necessity for a change in the situation in the future by abandoning for the use of the great liners the berths which necessitate the navigation of restricted and inadquate waterways, by the re-establishment and new development of the deep-water ocean port for the use of the future great ship units at new locations, where there is always deep-water access irrespective of tidal variation, where there are no limitations on size or character of piers or wharves, where ample railroad access can be obtained, and as close proximity as possible to

the main port entrance. In New York Harbor this would point to a future port development outside the Narrows.

The speaker referred previously to the limitations imposed by the Suez and Panama Canals. The Suez Canal when built provided only a depth of waterway of 26 ft. 2 in., with a bottom width of 72 ft. Work has steadily been in progress, for the 50 years since the Canal was built, aiming to deepen the waterway to 40 ft. and to widen the canal so that vessels can pass at any point, which work is well advanced and is in continuous progress, but, even at the present time, the Suez Canal offers a very distinct limitation on the sizes of vessels operating to the East.

At the present time, the Panama Canal is undoubtedly adequate for any vessels in sight; but if the same conditions are experienced in the development of shipping that have been experienced since the building of the Suez Canal, it may be contemplated whether in 50 years from now the provisions of the Panama Canal will be adequate to care for the then character of vessels to be operated. At that time it might be considered whether the locks and dams could not be raised so as to increase the water rather than to excavate to a lower level, as there is little likelihood of converting a lock canal into a sea-level canal once it has been built.

The presentation by the author of various methods of dredging might very advantageously be amplified somewhat in the question of disposition of the spoil excavated. The ease or difficulty of this operation is affected so greatly by local conditions that it must be considered principally as to general methods, and yet the resultant cost of dredging is affected very seriously not only by the actual transportation of the spoil, but also by the delays which may react on the continuous service of the dredging machines of whatever character used. Any port development project which couples with it a reclamation plan is obviously most desirable. The author has referred to the hydraulic method of spoil disposal which unquestionably is most efficient and desirable wherever it can be adapted, particularly in conjunction with reclaiming land. In the ordinary port improvement, it is seldom that a conveyor scheme, such as was used in the Suez Canal, for handling dredged material can be utilized, except in the occasional digging of a canal or narrow channel, so that to the largest extent one is thrown back on the necessity for using dump scows, which may be self-propelled or towed.

The combination of the dredging unit with its own self-contained, sea-going dump scow in one hull ("sea-going hopper type"), is commonly used in open seaways on account of the inability to hold scows alongside or to maintain hydraulic pipes to points ashore in a rough sea. The type, however, is undesirable for harbor work, as the continuity of the dredging operation is badly interfered with when the hoppers are filled and anchorage has to be broken to permit the most expensive part of the plant—the dredge itself—to proceed to sea or to dump, with the consequent serious loss in idle labor and lack of continuity in service. At the same time, some other means, not yet developed, is badly needed to meet these conditions of service.

When the distance to the dump is long, a combination plan for dumping at a station where material can be rehandled for reclamation use, has frequently been worked advantageously.

Any means, of a permanent character, to anticipate the increase in sizes of ship units—whether in the ports of the North Atlantic, in order to provide for the

enormous vessels which are now in sight, or in the ports of this and other countries at which a comparatively smaller type of vessel will be used—represents the outlay of vast sums of money. The world ports at which the super-provision for big vessels must be made are probably not a very great number, but each country in its own way must face the conditions indicated by navigation if they are to hold their future trade and commerce. Obviously, therefore, the more consideration that can be given to this subject the better, tending to educate and influence both Governments and people, so as to lay out for the future some comprehensive development for each port suited to each one's individual needs, in order that as the need is reached in each locality a definite plan for execution may be advanced. The engineer has still a future in sight.

EDWARD BURR,* M. AM. SOC. C. E.—It is desirable to correct any possible misapprehension which might be left by Mr. Davies' discussion regarding the depth of the channel in the Port of New York. There is no doubt as to the existence of a channel 40 ft. deep from the sea to deep-water frontages in the harbor, that is, to the frontages on the North River and on Bay Ridge Channel. The *Leviathan* went to sea in 1918, drawing 43 ft., and in that same year there were other departures of ships drawing more than 40 ft. In 1919, also, ships drawing 40 ft. and more left the port.

There has always been a natural depth exceeding 40 ft. at mean low water from the sea to and through the North River, except only across the ocean bar between Sandy Hook and Coney Island. This natural depth had not at all points the width necessary to accommodate, with the greatest convenience, the deep-sea traffic of this port where much congestion results from the intensity of local traffic, and, moreover, a part of the important frontage was obstructed by shoals of less depth than 40 ft. Present plans contemplate a depth of 40 ft. for a width of 2 000 ft. from the sea to and through the North River, with the same depth on the Manhattan front from the Battery to 61st Street, at Hoboken on the New Jersey front, and for a width of 1 000 ft. through Bay Ridge Channel. Under these plans Ambrose Channel across the ocean bar was completed in 1914, and has since required no dredging for maintenance; some dredging has been done in the Upper Bay where the available 2 000-ft. channel has been narrowed somewhat by the encroachment of the deep-water anchorage area; the narrow section between the Battery and Canal Street, where the natural width for the 40-ft. depth was only 1 000 ft., is in hand for early completion; the Hoboken front has 40 ft., and the Manhattan front from 19th to 61st Streets is well advanced to completion with the present depths sufficient at all docks for the steamers using them. The work has been and is being done by dredges of the sea-going, hopper type, except for one contract immediately adjacent to the Manhattan front where conditions made impossible the use of the hopper dredges, and has all been original dredging, that is, no maintenance dredging has been required up to this time.

The impressions of Mr. Davies in regard to lack of depth may have resulted from these operations for increasing the width of the channel, and generally from lack of appreciation of the character and value of the operations of the dredges used. In any case there has been a channel depth of 40 ft. since Ambrose Channel was completed in 1914, and that depth is available to all docks in use by vessels of

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FIG. 1.—MATERIAL DREDGED FROM CRAVEN SHOAL BY U. S. DREDGE *Atlantic*.

FIG. 2.—MATERIAL DREDGED FROM CRAVEN SHOAL BY U. S. DREDGE *Atlantic*.

the deepest draft, including the North River and Bay Ridge Channels. Incidentally, it may be added that work is in progress on a 40-ft. depth of channel through the East River also.

Mr. Robinson's paper is not only timely, but it is of extreme value by reason of his breadth of treatment of the subject. He has touched on three of the elements pertaining to the economics of deep-sea transportation, *viz.*, the size of the ship, depth of the ship channel, and the mechanical methods for securing that channel. A fourth element, that of the port terminal facilities, he touched on, also, although not in his paper.

The author's views on all matters pertaining to dredges are recognized by all as having the highest value. Supplementing those views, some notes on the operation of sea-going, hopper dredges in New York Harbor may be of interest. Ambrose Channel, with a depth of 40 ft., and a width of 2 000 ft., was excavated with sea-going dredges of the hopper type; 66 000 000 cu. yd. were removed at an average cost of 6.12 cents per cu. yd., the distance to the dump being on the average 10 miles. Of this quantity, 47 428 000 cu. yd. were removed through the operation by the United States, on force account, of four dredges built for this work, the average cost being 5.36 cents per cu. yd., including all expenses, but with credit for the remaining value of the dredges on the completion of the work. The material was sand varying from fine to coarse, with some small gravel and varying quantities of mud up to 20 per cent. A large number of dumps of stone were encountered and increased the cost of the work.

When Ambrose Channel was completed, there remained on hand two of these dredges for which there was then no employment for ocean-bar dredging, and they have since been used on the improvement of the interior channels of the port with marked success in two important respects, economy and non-obstructiveness to traffic. Every dredge or other machine working at a fixed position in a channel is an obstruction to traffic, which is greatly increased if moorings are necessary, as in swift tidal currents, and the congestion existing in the main channels of New York Harbor, complicated by tidal conditions, creates a most serious problem in the prosecution of operations requiring the use of such machines. Collisions are frequent under these conditions, with resulting increases in the time and cost of prosecuting the work. The hopper dredge is free from these disadvantages and operations for five years in the main channels of the inner harbor, all the year round on a twenty-four hour basis, have resulted in no complaints from navigators and few accidents, none of which has been serious. Delays from ice and wind are eliminated almost entirely, and delays from fog occur only when it is dense enough to suspend all harbor movements.

Although these dredges were designed to handle the sand of the ocean bar, they are used with marked economic success in removing mud, such as the Hudson River silt, in the inner harbor, and have been successfully used on clay or gravel and cobbles. Figs. 1 and 2 show material from Craven Shoal in the bins of the dredge *Atlantic*. In material suitable to such a dredge and in localities permitting the handling of a sea-going steamer (the dredge *Raritan* is 290 ft. long and has a bin capacity of 2 884 cu. yd.), these dredges excel in economy all types of plant except pipe-line dredges, being effective both as excavators

and as transport units when the dump is at a distance. As excavators they are not far behind the best records of pipe-line machines. The *Raritan* in the course of about 6 years dredged 5 565 loads from Ambrose Channel at rates varying between 1.4 and 10.1 cu. yd. per 20-in. pump per minute of pumping time, and averaged 6.04 cu. yd. per pump per minute of pumping time (362.4 cu. yd. per hour per pump and 725 cu. yd. per hour for the dredge) for the entire job under all the varying conditions of dredging, weather, and material, including some dredging at anchor, all of which militate against the dredge. In Hudson River silt the *Raritan* has averaged over 600 cu. yd. per pumping hour for considerable periods. These figures are "bin measurement", and neglect the material dredged and carried overboard in the overflow which, although small in sand or other heavy material, is relatively large in light material, and should be credited to the dredging equipment considered merely as an excavator.

As transportation units when the spoil must be deposited at a distance they are also most efficient. Notwithstanding the relatively high overhead, material, and labor costs resulting from greater first cost and operating cost as compared with the customary unit composed of a tug and one or more dump scows, the transportation or disposal cost from points in the Upper Harbor for hopper dredges is less than 50% of the cost of scow disposal. This results from seaworthiness and greater speed (9 to 10 knots) and consequent practical independence of tide and weather conditions. Figures of actual cost are reliable only for the period covered by them, on account of constantly and widely varying prices of labor and material, particularly coal, but the disposal costs for two hopper dredges averaged about 0.4 cent per cu.-yd.-mile in 1919. The total costs per cubic yard vary, of course, with market conditions, but have been considerably less than prices bid for favorable contracts in the same localities and material. The balance in favor of hopper dredges is still more pronounced on maintenance work, or other thin digging, for which they are peculiarly well adapted.

These notes cover the dredging operations which, perhaps, were observed by Mr. Davies, and were the basis of his remarks. A full knowledge of them is necessary to appreciate the effectiveness of the methods used. Up to the present time, maintenance dredging has been necessary only in the relatively shoal and narrow channel along the Weehawken-Edgewater front and in the southerly part of the Hoboken front. Sufficient time has not yet elapsed to determine the extent to which shoaling may be expected, following dredging in the Inner Harbor. Shoal areas have resulted from natural causes and the same causes may be expected to tend to reproduce them, particularly if it is borne in mind that the material (silt or mud of varying consistencies) encountered in the Upper Bay and the North River is easily eroded. In any case this difficulty can be met only by re-dredging, and no machine equals a hopper dredge for such work.

There has come to the speaker's attention another item of interest which he is at present inclined to attribute to the operations of hopper dredges. Ambrose Channel follows generally the location of a minor natural channel with the usual characteristics of an ebb-tide channel, including minimum depth at the outer or sea end. Based on the usual tendency of natural conditions on ocean bars to recur, it was expected that some shoaling would occur in the dredged channel as

a result of varying tidal currents, some of which did not run true to the channel location. The work was fully completed in 1914 and no dredging has since been done in the channel. A survey was made in 1919 to ascertain conditions, and comparison of results with the last preceding survey of 1915 shows that not only had there been no shoaling, but that depths had generally increased in the channel and over the adjacent shoals. This condition was so contrary to expectations that all controlling data were carefully verified to assure correctness.

Other channels across the ocean bar, deepened by hopper dredges, have also shown good qualities of maintenance. Gedney Channel, the outer section of the principal entrance channel prior to the opening of Ambrose Channel, has well maintained its dredged depth of 30 ft. A portion of Coney Island Channel was dredged with a clam-shell machine and refilled within a year, but the same locality after re-dredging by a hopper dredge has also held its depth unusually well.

Hopper-dredge operations result in the removal throughout the length of the cut of a series of shallow furrows, a few feet wide and a few inches deep, so that a trench on the bottom is gradually developed of full length, but progressively as to depth and width. Results up to the present time on the ocean bar lead to the surmise that this process results in a slow diversion of tidal currents so that they run more nearly true with the channel and aid in its maintenance or, in other cases, nullify the tendency to shoaling due to cross or oblique currents. These experiences extend through five years or more and although they are of too short duration to predict with confidence their indefinite continuance, they nevertheless are sufficient to indicate a marked additional advantage from the use of hopper dredges, and one well worthy of consideration for any work for which such dredges are adapted. It does not necessarily follow that similar results are to be expected in the lighter material and different current conditions of the Inner Harbor, and in certain channels of relatively narrow width shoaling has already occurred, but there is some reason to expect it where the dredged areas are of sufficient width and continuity to affect currents materially.

For years the problem for solution for the maintenance of the efficiency of the Port of New York has not been the main channels of the port. The provision of ample and efficient terminal facilities is far more important, and is far more backward in its development than the channels of entrance and exit. The U. S. Corps of Engineers has been referred to several times in the discussion of this paper, and it has been very gratifying to feel that on the whole those references have been not uncomplimentary. In one respect at least the Corps has been up to date, and that has been in the provision of channels of satisfactory depth in this port, and in the other ports on the Atlantic Coast of this country.

The Ambrose Channel project with a depth of 40 ft. at mean low water was recommended by the Corps of Engineers and authorized by Congress in 1899, a depth of 35 ft. having been previously under consideration. In 1907 it was partly opened to navigation with a depth of 35 ft.; in 1909 the depth was 37 ft.; in 1910 the depth was 40 ft. for a width of 1 000 ft., and early in 1914 the channel was fully completed.

At the present time the statistics of departures from this port indicate that of ships drawing 27 ft. or more, only 5% have exceeded 35 ft. in draft, notwithstanding that there has been since 1910 a 40-ft. channel at mean low water; and

the departures of ships drawing 30 ft. or more are only about 27% of the total departures exceeding 27 ft.

These statistics are not brought forward as an argument that greater depths are not necessary, but as evidence that the channels provided for the Port of New York up to the present time have been ample to meet the needs of the port, while it is common knowledge that the facilities for handling the traffic of the port after its arrival at the terminal have been and still are far from sufficient or efficient. If the past may be used as a criterion, the future may be looked forward to with confidence, and it may safely be expected that the channels of the Port of New York will be enlarged if and when its commerce requires greater channel capacity.

J. H. H. MUIRHEAD,* M. AM. SOC. C. E.—A point has been raised on the other side of the Atlantic, namely, that Christopher Columbus was more able to start from Europe and deliver goods to the American public than any steamship captain of to-day; the point taken is that after the vessel is docked the trouble begins. Of course, it has been said that unloading a cargo is not engineering, but the speaker maintains that it is.

Some years ago he came into the Port of New York on a ship which had on her deck magnificent hydraulic machinery with which to unload the whole ship in a short time. The port regulations allowed her to pass out her passenger baggage, and that was all that was permitted. As far as the freight was concerned, the machinery had to be closed down, and the ship had to be unloaded of her regular freight by hand. The speaker understands that this regulation has now been modified.

Why do engineers neglect to speak of those points which affect the whole Profession? In this connection, the speaker will relate an absurdity which illustrates this point, showing the ignorant man interfering with the engineer. Some of the greatest engineering schemes have been spoiled by political interference of an entirely personal and unjustifiable character.

It happened in Scotland when the speaker was an apprentice that it was his duty to hold out the end of the plans which he had drawn while the Chief Engineer showed them to a committee. The members of the committee were of the town-councillor type; they were not particularly learned men. One of those men wanted to "spring something", and he said to the Chief Engineer in Scottish dialect, "Now, Mr. Miller, I have been considering that at some future period it might be desirable to put cranes on the dock, and some provision ought to be made for that now." The Chief Engineer explained that that was not at all necessary, because the dock walls went down to bed-rock; everything was founded on rock, and no better foundation could be had. When they were on the solid rock there was no need of any further provision. A few days after the man came back, and said, "The Committee has considered the crane matter and sent me to tell you that you have got to make provision for putting in cranes at a future period." The Chief Engineer said: "But, my dear sir, I cannot do anything of the kind; the Lord Almighty has already provided rock for that purpose"; to which the man replied, "Now, Mr. Miller, the Committee

* New York City.

does not give a —— what the Lord Almighty has done; it has decided that this is the duty of the Chief Engineer."

Although all engineering schemes are subject to the interference of those who simply wish to show authority, to practice doubtful economy, or to conserve minor interests, the speaker fully believes that harbor improvements get more of this than their share. In spite of believing fully in the Department of Docks and Ferries as the most efficient of the New York City Departments, the speaker considers the New York docks quite inadequate for the present business, and totally lacking in preparation for the future. Unloading machinery is entirely wanting; very few of the docks are even provided with a railroad track, and none of them has overhead traveling cranes with which to stack the arriving cargoes and load them on trucks for distribution throughout the city.

Referring to the statement that other nations should keep up with the United States, in the matter of port equipment, the speaker would say that even in undeveloped South America there are locomotive cranes on tracks outside of the sheds in many harbors, and all the so-called modern sheds have two inside, overhead traveling cranes of a design similar to those used in American machine shops and engine factories.

It has been claimed that properly to equip the docks with unloading machinery would take the bread out of the mouth of the workingman. The workingman is not going to live for ever; his son is not going to be a laborer, he will have an education, will be able to control machinery, and be worthy of an increased wage for doing so. The speaker feels certain that the American workingman of even 10 years hence will flatly refuse to "jackass" cargo.

WILLIAM J. WILGUS,* M. AM. SOC. C. E. (by letter).—The outstanding feature of the author's papers, both the one now under discussion and the previous one to which he refers,† would seem to be summarized in the opinion he quotes from the report of the British authorities to the effect that the trade of the world, under the sharp competition that is to rule in the future, naturally will follow the cheaper and better routes, and that the development of such routes, in the last resort, will depend on the increase in the size and draft of sea-going vessels and, consequently, on the improvement of ports to serve them properly. In a word, the maritime nation which can offer the lowest freights and best service overseas, through the superior development of ships and ports, will have a pronounced lead over its competitors, just as in land transportation industrial growth follows the highly efficient railroad in preference to the one having under-capacitated rolling stock and inadequate terminals.

Therefore, it behooves Americans to awaken to this situation and develop their principal ports so that they may be in readiness to welcome with open arms the largest cargo carriers of which there is serious consideration, as, for instance, vessels of the kind mentioned by the author having a length of 660 ft., a loaded draft of from 36 to 38 ft., and a dead weight capacity of 25 000 tons. For the proper accommodation of a ship of this size the depths of channels and slips, as he so well states, should be not less than 40 ft., with provision for future deepening to 50 ft.; track facilities should exist for the direct interchange of mail and freight between

* New York City.

† *Journal*, Eng. Inst. of Canada, February, 1919.

ship and car; electric gantry cranes and other mechanical devices should be provided for the speedy handling of materials of widely varying weight and bulk; slips should be generously proportioned for the berthing and maneuvering of ships, tugs, lighters, canal barges, and other craft; and the width of piers should be ample for the receiving, forwarding, sorting, and temporary storage of cargo and for the location of the cranes, tracks, and vehicular driveways necessary in connection therewith. The aim of such an arrangement is to hasten to the utmost the release of ships, cars, and motor trucks; to minimize as far as possible the use of manual labor; and to dispense with the need for ship tackle for the reasons so clearly explained by the author.

The question of pier widths is of special importance as bearing on the fundamentals of port design. A ship like that under consideration will take up, say, 700 lin. ft. of string-piece, approximately equal to 36 long tons of dead weight capacity per linear foot. Opinions will differ as to the factors to be used in determining the superficial area of shedded "transit" space that should be provided for a tonnage of that amount. Some consider that such space should be made sufficient to accommodate the entire cargo, figured at 60 cu. ft. per ton of actual cargo or, say, 50 cu. ft. per ton of the dead weight capacity of the ship, equal to 1 800 cu. ft. of goods for which shedded space should be provided; and they also consider that goods, with mechanical tiering, can be conveniently piled to a height of 12 ft., or, with suitable provision for aisles and passages, to an average height of 10 ft. for the entire area, thus making the required shed width 180 ft. Others believe that one-quarter of the cargo may be immediately evacuated from the pier, leaving three-quarters, or 27 tons per lin. ft. of string-piece, which would require shedding; and that this amount in conjunction with the usual ship-ton figure of 40 cu. ft. per ton and an average height for tiering by hand of 4 ft. (5 ft. on the actual space occupied), would make the required width of shed 270 ft. It may be said that a fair compromise between these extremes is 200 ft., to which, of course, there should be added a suitable width for outer tracks and working space between the edge of the pier and the shedded area, and likewise ample interior widths for inner tracks and motor-truck driveways.

In the case of the double-sided pier this would mean an aggregate width of not less than 550 ft., the lower floor of which would be used for inbound and the upper floor for outbound cargo. Even though a ship of only half the capacity under discussion should be adopted as the maximum for which the port should be planned, it is evident that the pier width should be not less than 340 ft. It is assumed, of course, that the crane equipment, track layout, and driveways would have a proper relation to the length of pier, as the greater the number of ship berths per pier the greater the need for efficient means of handling and transporting cargo between ship and shore in order that operations may be continuous. Should overhead warehouses be provided on the pier, there would be need for even greater widths than those mentioned because of the necessity for additional serving tracks and driveways. These figures are used in a general sense and are to be taken as merely indicating the need for the adoption of liberal pier widths in the planning of modern deep-water port improvements.

In this connection, it should be borne in mind that the investment in port terminals is a comparatively small fraction of the aggregate investment devoted to

the combined ocean and rail movement of freight. Therefore, a liberal expenditure for the perfecting of the lesser part is fully justified if substantial benefits therefrom will accrue to the plant as a whole, in which are included ships, cars, and motor trucks, as well as the shore facilities in general.

With the need proven for big ships and adequate terminals to serve them, and with a decided move in that direction already taken by Great Britain and her Dominions, and in South America and the Orient, is it not natural to inquire what is being done by the United States to put itself in condition to meet that situation?

On the Atlantic Coast it is only at New York City that we have a 40-ft. channel and this, unfortunately, is now available for only a little more than one-half of 1% of that port's 771 miles of water-front. Of this meager percentage, only one-half is in direct contact with the principal rail carriers that extend to the interior of the continent. Moreover, a considerable portion of the deep-water frontage is devoted to uses other than those of legitimate water-borne commerce. Not only is there limited harbor frontage for big ships in liaison with the land carriers but, apart from the projected Claremont Terminal of the Lehigh Valley Railroad, there is as yet nothing planned, as far as is known, in the way of spacious and well equipped modern piers and accessories, properly adapted to the prompt handling and release of large ships, cars, and motor trucks. It is true, according to press accounts, that of the great transatlantic companies several intend to build new piers on the westerly side of the Hudson River; but it is reported, as in the case of the development proposed by the City of New York on Staten Island, that advantage is not to be taken of the proximity of trunk-line railroads to provide tracks on the piers and adequate shedded "transit" areas, in consequence of which the rail carriers, and, therefore, indirectly the people of the country at large, will continue to be burdened (*a*) with the wasteful practice of rehandling freight by lighter and truck between the termini of the rail haul and ship; and (*b*) with highly objectionable delays to cars caused not only by this double operation, but also by the enforced use of such cars for the storage of freight that should be provided with reservoir space on the piers.

It may well be asked what are the underlying causes for this unhappy situation at the Nation's principal gateway.

One of the causes is unquestionably the provision in the seaboard rate that imposes on the rail carrier the duty of delivering and receiving cargo freight at the ship sling, coupled with a denial to him of a voice in the planning and administration of the common ground on which the two agencies of transportation meet. The water carrier feels that it is no concern of his that the railroad is subjected to wasteful costs and inordinate delays to cars and lighters, and, in consequence, he is loathe to shoulder the burden of providing more efficient shore facilities from which he considers the other party to the transaction will reap the larger share of the benefit. On the other hand, sharp competition for seaboard traffic, and the concentration of steamship terminals on the easterly side of the Hudson River, where lighterage deliveries have been made necessary by reason of the absence of universal direct track connections, have combined to stifle protest by the rail carriers against this unnatural situation from which they, and, therefore, the public, suffer.

Another cause is the short-sighted municipal practice of permitting the lessee, the ship owner, to dictate the character of the improvement, on the basis of a rental that yields a return on the city's investment. The result is the creation of piers at the least possible cost to the shipping interest regardless of the injury done to the land carriers, both rail and motor truck. A narrow policy of this kind brings its own punishment in the stimulation of rival projects for diverting trade to other routes and in the alienation of Government aid for deepening harbor channels. It is "saving at the spigot and wasting at the bunghole."

It will be seen that there are three parties to be considered, the communities which cluster about the port, the life of which rests on the continued flow of trade through its portal, the carriers which need the best of modern terminal facilities if they are successfully to meet competition, and the Federal Government which now, more than ever, has a vital interest in the providing of efficient facilities for its vast merchant marine and in the economical operation of the railroads. They should get together to bring about: (*a*) the revision of the rate structure and the methods of paying for the use of shore facilities, in such manner as more equitably to distribute the burdens and benefits of interchange between the rail and water carriers; (*b*) the adoption and carrying out of plans that will be conducive to economies in operation and quick release of ships, cars, and motor trucks; and (*c*) whole-hearted co-operation by the Federal Government in these reforms which will mean so much for the country as a whole.

When happily this may be brought about, as, for instance, through some such instrumentality as a Port of New York Authority like that which has proven to be so successful at Liverpool and London, New York City shall indeed have need for the perfected dredging machinery referred to by the author.

EDWARD F. HAAS,* M. AM. SOC. C. E. (by letter).—In reading Mr. Robinson's discussion of four types of dredges, the writer was surprised that so little space was devoted to the clam-shell type. In an 11-page article the subject is dismissed with a single paragraph, in which the author states:

"It is exclusively adapted to soft material and finds a wide range of application outside the dredging field, as in handling coal, ore, and sand. As a dredge, it is specially adapted to such work as cleaning out slips, etc., in confined locations, but for hard material or work in the open, the other types are better."

It may be of interest to say that in California the clam-shell dredge is the predominating type. There are a number of hydraulic machines used, including the sea-going type operated by the Government, but very few dipper dredges, and these are of small capacity. In San Francisco Bay and its tributaries, there is not much rock excavation and only a limited quantity of hard material, so that the clam-shell is peculiarly adapted to this locality. There are more than fifty clam-shell dredges in this territory, with an output of several hundred thousand yards per day.

It is a mistake, however, to say that the clam-shell is limited to soft material, or that for work in the open the other types are necessarily better. The writer has successfully used the clam-shell type for scow work, having twice dredged the ship channel through San Pablo Bay. At this point the depth was not extreme, reaching about 40 ft. at high tide, but the current was swift, and the location was

* San Francisco, Cal.

exposed to strong winds and wave action. Both spuds and anchor lines were used.

On another occasion the writer had to remove a sea wall which had been pushed out into the main channel. This sea wall was composed of rock weighing from ½ ton to 4 or 5 tons, which had to be excavated and put into barges. By using a 6-yd. clam-shell bucket, no difficulty was experienced in doing this work, and many thousands of yards were removed.

In the removal of Blossom Rock, in San Francisco Bay, which was situated directly opposite the Golden Gate and exposed to the swell from the ocean, as well as swift current and strong winds, the entire yardage was removed, after blasting, by a clam-shell dredge. In this case the material was picked up and swung out into deep water without having to be loaded on barges; doubtless it could have been handled by dipper dredge, although the reach necessary to get into deep water would practically have required double handling, or else loading it on barges. It is quite certain that the clam-shell was the most suitable type for this work.

The principal use for the clam-shell machines, and the use through which that type was perfected on the Pacific Coast, is for the purpose of building dikes. In the early stages, hulls about 40 by 80 ft. were used, with a boom of from 100 to 120 ft. to swing about a 2-yd. bucket. As more land was reclaimed, and it became necessary to build larger levees, the type of machine has been improved, and the size increased. The largest machines have steel hulls about 70 by 140 ft., carrying booms of about 240 ft., with buckets of from 5 to 6-yd. capacity. These buckets are equipped with teeth and weigh from 10 to 12 tons. Even the smaller buckets are equipped with teeth for hard digging, and are successful for that purpose.

In the work of levee construction throughout the delta of the Sacramento and San Joaquin Rivers, it is necessary to excavate earth materials of all kinds, from peat, mud, sand, and stiff clay up to hardpan. In places the levees are of such height and width that it would be impracticable to use dipper machines on account of their lack of reach.

The writer doubts whether dipper machines could be constructed to compete in height and distance of throw with the modern clam-shell dredge with 240-ft. boom. Furthermore, the ease with which the clam-shell is manipulated is a decided advantage. It is not necessary to hold the hull stationary for digging or swinging, which eliminates the rigid spud arrangement. The hull lists with the load, the spuds moving freely in their casings. Depths of 60 ft. are easily dug, and it is possible to dig much deeper—the only limit being the length of rope necessary to reach bottom. At great depths it becomes necessary, of course, to work with lines instead of spuds.

Considering the range of work for which the clam-shell dredge has been used, as outlined in the cases cited, it is believed that the author has hardly been fair to this type in relegating it to soft digging in protected places.

FRIEND P. WILLIAMS,* M. AM. SOC. C. E. (by letter).—The author has taken up an especially timely subject in view of present-day shipping conditions. With especial reference to dredge design, the writer wishes to furnish some data to show the status of the dredging efficiency of some present-day dredges. Although

* Albany, N. Y.

Mr. Robinson's paper relates chiefly to deep excavation in connection with ocean harbors, the writer will submit some data on shallow dredging, particularly in view of Mr. Wheeler's remark in his discussion that "probably as much time is devoted nowadays to the repair of dredges as to actual dredging".

Mr. Robinson probably had improvements in machinery in mind in preparing the paper, and assumed effective operation. The writer believes considerable efficiency can be gained in adopting correct operating methods. Attention to suggestions in this direction, as well as in design, he believes will be helpful in securing effective operation at low cost under varying conditions.

As to the effectiveness of present-day design, especially as to losses in actual running time, the log of operations of a 20-in. hydraulic dredge operating at a depth of 12 ft., such as shown in Fig. 3, indicates that for considerable intervals the dredge was operated for approximately 20 hours per day, with the exception of shut-downs for washing the boilers, which occupied about 2 days at 3-week intervals.

In the construction of the Barge Canal during the past fifteen years, a variety of dredging plant has been used, some of it of makeshift order. To a large degree, the adoption of makeshift plant was due to the fact that the extent of future work that could be secured for a dredge being built by a contractor, was uncertain. Under these conditions, it was only natural that makeshift methods should be used. However, in the case of large contracts, there were sufficient inducements, and as the work progressed a number of high-grade dredges were constructed.

As far as securing plant that will stand up under continuous operation is concerned, the writer believes that the designer in the case mentioned has definitely succeeded. If the log of operations in this respect for deep harbor dredges approximates the record in question, Fig. 3, it would be helpful in gauging the efficiency of the best present-day dredging practice.

It is to be expected with plant like dredges, which usually do not command sufficient work for continuous operation, that there should be less settled design and less economical operation than is possible for steam shovels which command considerable continuous work. For this reason it is especially valuable that the author has discussed this subject of dredging and outlined some of the improvements, and also the possibilities, in dredging plant. The writer believes that all those who are interested in dredging should be on the alert to make improvements, and should not be satisfied with present-day records, although some dredges under particularly efficient management have shown up especially well.

In connection with the dredging record before mentioned, Fig. 3 shows the log of the dredge *Lyons*, which gives in detail the actual time of operation and the causes of delays. The policy in connection with operating this plant is to carry a very large number of supply parts, and also to maintain a good machine shop. It is a routine matter for each shift on going to work to inspect the plant, tighten up loose bearings, and make any repairs that will keep the machinery in running order. This is found to be very effective in cutting down repairs.

During the months of September, October, and November, shown on Fig. 3, this dredge was working in hard material in the Clyde River. Much of the

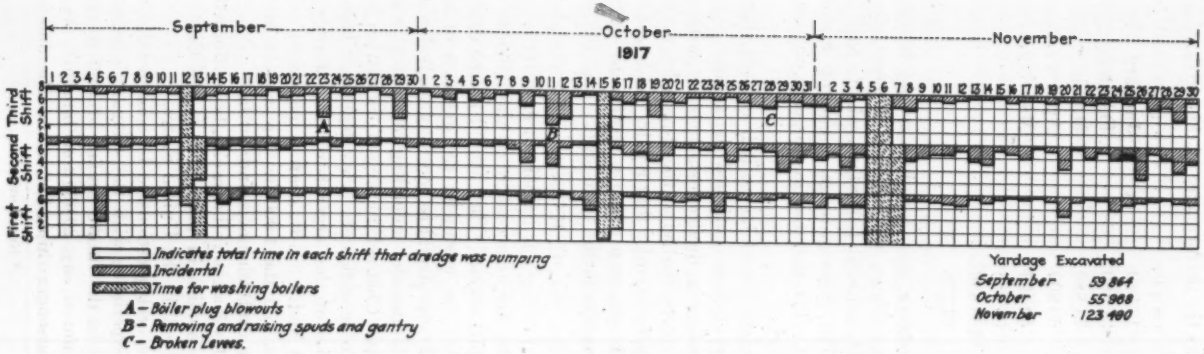

Diagram Showing Number of Hours in Each Shift that The Hydraulic Dredge **LYONS** was Operating During the Months of September, October and November, 1917.

FIG. 3.—LOG OF PERIOD TYPICAL OF PERFORMANCE OF HYDRAULIC DREDGE *Lyons*.

material had to be drilled and blasted and consisted of clay, gravel, and soft shale. The following shows the performance of this dredge for several months:

1917	Cu. yd.	1918	Cu. yd.
May	12 360	January	50 670
June	31 460	February	49 770
July	73 880	March	28 490
August	60 800	April	90 380
September	59 864	May	29 440
October	55 988		
November	123 490		
December	34 230		

The greatest record, however, was made by a second dredge on the same work—the 20-in. hydraulic dredge *Clyde*, which is not as powerful as the dredge *Lyons*. The *Clyde* in August, 1911, removed 480 681 cu. yd. of "pay" material in the north end of Cayuga Lake. The gross yardage, including material outside of "pay lines", was 633 561 cu. yd. The material in this case was sand and silt, the most favorable for hydraulic work.

In operating these dredges for 24 hours, it is evident that, excluding the time for washing the boilers, operation continued the greater part of the time. It is also shown that in operating a dredge continuously with three shifts, repairs should be made and the boilers washed at the time when most needed, rather than continuing to run inefficiently in order that repairs can be made during a particular shift, as would doubtless be the case where a dredge has operated two shifts, with a tendency to throw the repairs in the third shift.

CHANDLER DAVIS,* M. AM. SOC. C. E. (by letter).—The author has called attention to the necessity of improving our harbors to meet the requirements of modern ships. These improvements are as necessary on the Atlantic seaboard as they are on the eastern shores of this ocean. They should not be confined, however, to the deepening and widening of the channels; the question of installing mechanical appliances to handle the cargoes should be carefully studied and machinery designed which will expedite the loading and unloading of ships at a minimum cost.

At the time the Ambrose Channel, in New York Harbor, was dredged, a depth of 40 ft. was considered to be sufficient. This depth had been adopted as a standard by the port authorities of the larger harbors of Europe, but at that time a 700-ft. ship was a novelty and a 325-ft. draft a maximum. To-day a 900-ft. ship is no longer a conjecture, and the 1 000-ft. ship may be laid down at any time. Such ships require deeper draft channels and berths.

The largest vessels to-day load to full capacity, being limited only by the depth of the channels. This fact applies in particular to the City of New York which happens to be the port of call in the United States.

There are, however, several questions to be considered. Has the war affected shipbuilding? Has the experience with the submarine changed the trend of the naval architect? Will he in the future confine himself to smaller ships, limiting their tonnage to a maximum of, say, 20 000 tons burden? As a matter of fact, the very large steamers are passenger ships, few in number, and anything but econom-

* New York City.

ical to run. The shipbuilding programme should be studied before deciding what improvements are required.

In considering these improvements, freight-handling machinery is an important factor. The harbor of New York is an example of a port where the installations for loading and unloading ships are primitive and undeveloped. Coal-handling machinery is lacking, and most of the ships are bunkered by hand. Although the International Mercantile Marine has a very efficient and economical portable electric hoist, its use is confined to the lines controlled by this company, and cranes, derricks, and other apparatus for handling cargoes are lacking. The lay-out of the harbor may have something to do with this, as well as the fact that considerable freight is handled by lighters. This method of handling freight should be replaced by a marginal railroad which would connect the docks with the railroads centering in New York City. Several designs for such a road have been considered from time to time, but none has been feasible. They were designed as elevated roads, practically requiring a new lay-out of the piers and sheds.

Even if a marginal road was built, the installation of machinery must not be neglected, and the existing piers and sheds make this a difficult problem, one which must be solved for each case. It is usually customary when large improvements are contemplated, such as those in the Gansevoort and Chelsea Sections, to lease the structures before construction is commenced. Such a procedure simplifies the design of the cargo-handling plant, as the needs of each company can be taken into consideration and a plan adopted suitable for the business of that company.

It will be necessary, therefore, to build approach channels of sufficient width and depth to accommodate the shipping, design the piers and sheds to permit the installing of suitable cranes, hoists, and such other appliances as may be required by the cargoes carried by the vessels, reduce the handling of freight by lighters, and, finally, construct a marginal road which will connect the port with the railroad, thus enabling the immediate loading of the cars and reducing the cost of handling the export freight to a minimum.

A. W. Robinson,* M. Am. Soc. C. E. (by letter).—The author is much gratified that his brief paper has called forth so much useful discussion and criticism.

Maj. Dent's opinion as to the undesirability of large ships is one widely held, and the author has never advocated large vessels without stating, as a qualification, the necessary conditions for their successful use. The reason why larger vessels are not now used to any great extent is that these necessary conditions do not, as a rule, exist; but these can be created and should be created for main trade routes if traffic is to be carried in a wholesale and economical way. To assume, as Maj. Dent appears to do, that the cost of assembling, loading, or transferring a large cargo is greater than a small one, is to maintain that wholesale methods are more costly than retail ones. All depends, of course, on the suitability and economy of the methods used.

The American merchant marine is now receiving some very fine vessels, but for the most part and for the vessels built during the war period, it is now where the railroads were in the days of 20-ton cars, light locomotives, and no air-brakes.

Maj. Dent truly remarks that the present-day practice of hydraulic dredges follows the "rule-of-thumb" method in designing and curing defects that develop

* Montreal, Que., Canada.

to such an extent as seriously to discourage the engineer who wishes to work on a more scientific basis. Concerning his remark that the capacity of the pump and pipe line is, in tough materials, ordinarily much greater than the capacity of the cutter to loosen the material, the writer would say that his own practice is always to give the cutter a margin of capacity over the pump.

Mr. Wheeler's remark that some of the many expert ship designers could profitably give valuable aid to the less expert dredge designers is important. The trouble is that the ship designers lack dredging experience, and the dredge designers are seldom good marine men and they lack sufficient experience in maintenance and actual conduct of work. The result is that the former often burn their fingers and lose money in dredge building, and the latter do not rise to good marine standards.

Mr. Massey claims that Chicago is the center of dredge building for the United States. The class of work that is built so extensively in the Middle West and for developing the interior of the country, while excellent for its purpose, is quite different from that required for salt-water service at great depths, or for export abroad.

Mr. Massey's statement that the writer introduced lubricated pin joints on buckets for placer dredges built by the Bucyrus Company and that they were unsuccessful, is incorrect. The writer designed and introduced in 1888 the "close-connected" bucket now used largely for placer work, but lubricants were never used as they would be incompatible with gold saving. Lubricated joints for ordinary dredging purposes, however, have been used successfully for some years.

Mr. Ferris' discussion refers to details of dipper-dredge design as built by his Company, and defends his details on the score of conservatism, though the writer made no criticism of them. Without attempting to discuss details here, the writer would remark that many of the details which Mr. Ferris now defends were designed by the writer 30 years ago. It may be conservatism still to use them, but the writer now regards many of them as obsolete.

Mr. Fowler cites the vast quantity of coal required for a great ship as a reason for limiting its use. This is exactly one of the reasons for increase in size, because more cargo is carried with less fuel. Also he states the "hundreds of millions" necessary for creating deep harbors precludes their possibility. This overlooks the fact that 40-ft. depth is now the recognized standard in many harbors throughout the world, and that this depth has set the pace for all main trade routes.

The writer regrets that the tone of Mr. Fowler's discussion is derogatory to all such advances as are advocated not only by the writer but by many experienced authorities, and he regrets also that he decries "scientific analysis" and "office design" as resulting in work that requires to be rebuilt and cites his own experience in this regard. Mr. Fowler's experience is unfortunate, and he is not alone in this experience. That a dredge or any other work requires to be rebuilt or corrected is not a condemnation of scientific design, but rather of the ignorance and inexperience of the man who designed it. In experimental work, no doubt, some corrections and improvements are necessary, and are steps toward full development.

Mr. Fowler's remark that the improvements in dredge design advocated by the writer will scarcely be recognized after the dredge captain tries them out on the work is entirely gratuitous. As a matter of fact they have been tested out in

severe service and as now built have required no alteration. A solid laminated dipper arm of the writer's design has been six years in hard service and is still as good as new, and a spare arm has not to this date been put on. A large steel dipper dredge of the writer's design was shipped thousands of miles to a foreign port to do extremely hard service. It was built on time, ran its trial period without a break, more than fulfilled its guarantee, and now after several years of service is in first-class order and has required only trifling repairs and no alterations whatever. This shows what can be done by good engineering, good organization, a high marine standard of workmanship and not slavishly following designs of thirty years ago or even the dredge captain's dictum. It is precisely to help to overcome and avoid such a state of things as has been Mr. Fowler's experience that the writer ventured to offer his paper and urge these reforms.

Mr. Fowler also "takes exception to the author's statements about the Corps of Engineers." There is no mention in the paper of the Corps of Engineers. The writer gives place to none in his high appreciation of the work of the Corps of Engineers, many of whom are his personal friends.

Mr. Davies' discussion is a broad and valuable statement of the shipping situation. He recognizes fully, as some of the other speakers failed to do, that the large ship and the moderate and small ships all have their separate function. His statement, however, that "nowhere, except on either side of the North Atlantic, do the channels, canals, and ports permit of the use of any vessels larger than about 16 000 tons" requires some qualification. Most of the ports on the main trade routes of the world have now, or are approaching, a depth which will carry 25 000 tons or more. The deepening of the Suez Canal to 40 ft. has been actively going on for some time, and that depth is now practically attained, and it is the Suez Canal that sets the pace for the Eastern Hemisphere. As Mr. Davies well says, however, the subject is one that extends beyond simply the larger units and deeper channels into the questions of general port development, involving great terminal problems and the handling of freight and bunkering of fuel.

The writer is grateful to Col. Burr for his valuable contribution and for the data given on dredging in New York Harbor. The sand-pump hopper dredges in service have done work which compares favorably with any similar work in the world. The low disposal cost achieved by these dredges is due to the fact that they are primarily a disposal apparatus of large capacity, that is to say, a vessel carrying a large load of sand at fair speed and not encumbered with any great weight or cost of dredging apparatus. Unfortunately, this very simple and effective type is not suitable to execute the great amount of dredging which still remains to be done in New York Harbor, as in the East River channel, and in enlarging and extending the central limits of the harbor, which will require a much more capable and effective class of plant than is in use at present.

Mr. Wilgus has contributed a valuable review of the important question of terminals, and calls attention to the fact that only one-half of 1% of New York's water frontage is available for 40-ft. depth. No more important subject is before this Society than a discussion and criticism of these questions that affect the public welfare, not only of the chief city but of the country at large. This discussion has been taken up by the New York Section of the Society, and it is to be hoped will be carried on when the report of the Commission on the Port of New York becomes

available. Mr. Haas has drawn attention to the great clam-shell dredges used in California. These are of a special type, the closing levers being made solid with the shell of the bucket, and dispensing with the usual multiplying chains for closing. They are thus very simple and effective, and are used with long booms for levee work, etc. Mr. Haas also quotes instructive examples of wider use of this type for channel deepening, raising blasted rock, etc. The writer recognizes fully the great field of this type of dredge, and if he failed to give it due place in his paper, it was because the main subject was the economic one generally, and detailed description of plant was necessarily brief.

Mr. Williams makes a very good point when he says that considerable efficiency can be gained by adopting correct operating methods. Elaborate plant is useless without this, and a makeshift temporary plant, if sound in principle and detail, can often show wonderful results when well run in good hands. The data given by Mr. Williams of actual performance of hydraulic dredges on the New York Barge Canal are most useful, and show what can be done under favorable conditions.

Mr. Williams also reminds us that one reason why dredges as a class are not as highly developed as other lines is because they are of less settled design and usually do not command sufficient work for continuous operation. This is due to the contract system under which it is seldom possible to provide permanent plant, and under which contractors frequently construct their own plant without adequate engineering knowledge, or are governed by individual preference and local experience. It is also due to the fact that dredges are built in various places at infrequent intervals and are not sufficiently numerous to crystallize into a survival of accepted practice. Even builders in this line may be restricted in their practice and outlook to their own product, and have always the manufacturer's bias to use old or existing patterns.

In conclusion, the writer hopes that this paper and the very interesting discussions which it has elicited may be useful, and may promote co-operation and a willingness on the part of all engineers to learn from one another.

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Paper No. 1466

THE PROBABLE VARIATIONS IN YEARLY RUN-OFF AS DETERMINED FROM A STUDY OF CALIFORNIA STREAMS

By L. Standish Hall,* Jun. Am. Soc. C. E.†

With Discussion by Messrs. Allen Hazen, L. F. Harza, Robert E. Horton, Kenneth Allen, C. H. Pierce, and L. Standish Hall.

Synopsis.

Data on the variations in the quantity of the yearly run-off of rivers are susceptible to treatment by the theory of probability. The purpose of this paper is to study the actual variations in the measured yearly run-off of California streams in order to construct a probability cross-section paper on which, by means of a straight-line relation, the probable minimum and maximum yearly run-offs which are likely to occur once in a period of 100 years, may be determined for any stream from the available run-off data for that stream.

In connection with the study, the relation between the arithmetic mean, the median, and the most probable values of the yearly run-off were also established, at least approximately. The cross-section paper, constructed as a result of this investigation, and which was named by the writer "Hydraulic Probability Paper", can be used in estimating the relative wetness and dryness of individual years, in terms of the probable absolute wettest and dryest years. By using these indices of the relative wetness and dryness, the probable values of the yearly run-off can be estimated for streams for which there are few or no run-off records, and for as many years as run-off measurements have been taken on neighboring drainage areas.

Introduction.

The writer's attention was first called to this subject in reading the paper by Allen Hazen, M. Am. Soc. C. E., entitled "Storage to be Provided in Impounding

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Reservoirs for Municipal Water Supply".* In this paper Mr. Hazen used a probability cross-section paper constructed to conform with the integral of the probability curve. This curve, which has been derived from the results of experience to represent the facility of errors, is well known and can be found in any book treating the subject of Least Squares, therefore no further discussion of its derivation will be given here.

All the curves plotted by Mr. Hazen on his cross-section paper to represent the probable storage required in impounding reservoirs for separate years, were of similar shape, being concave upward. The writer has investigated a number of the California streams plotted on Mr. Hazen's cross-section paper to determine the probable variations in the yearly run-off, and has always found that the data so plotted will be represented best by a curve concave upward. These results led him to the conclusion that the variations of yearly run-off did not follow the law of the facility of errors. This conclusion is justifiable, as would be the one that data which when plotted on logarithmic cross-section paper did not lie approximately in a straight line, could not be represented by an exponential equation of the general form ($y = a\,x^n$). It is not the intention to infer that data so plotted may not be better interpreted than it might be otherwise. Similarly, probabilities can be better studied on Mr. Hazen's probability paper than would be possible on ordinary cross-section paper. However, as a curve is more difficult to project than a straight line, it is evident that if a cross-section paper could be developed on which the probable variations of the yearly run-off could be represented as a straight line, it would add greatly to the usefulness of the method. Such a cross-section paper will be developed later.

Principles of Probability.

The principles of probability are defined by writers on the subject of Least Squares about as follows: The mathematical probability of the occurrence of an event is the ratio of the number of favorable chances of its happening to the total number of chances, favorable and unfavorable, each being supposed to be independent and equally likely to occur. In other words, if a certain event happened only once, out of one hundred possible chances, the probability of its occurrence would be 1 in 100; however, if it had occurred ten times with the same number of chances, the probability of its occurrence would be 10 in 100, or 1 in 10.

The most probable occurrence is that which has the greatest mathematical certainty, or which occurs with greater frequency than any other.

When the total number of chances, favorable and unfavorable, of an event occurring are known, the exact probability of such occurrence in any specified way can be computed mathematically. However, if only a portion of the favorable and unfavorable chances are known, the probability of the event occurring in any specified way can only be approximated, the accuracy of the approximation depending on the number of favorable and unfavorable chances of the event's occurrence which have been observed. In order to determine the relative probabilities in the latter case, it is necessary to count the number of observations

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

of the event to be studied which occur in assumed equal intervals from some definite point, for example, the arithmetic mean. If the number occurring in each interval is divided by the total number of observations, the results will be the probabilities of the event occurring in each of the intervals chosen. The choice of the interval to be used depends on the total number of observations, and on the maximum variations of individual occurrences above and below the arithmetic mean. If the maximum variations are only 10% of the arithmetic mean, intervals of 1% of the arithmetic mean could be chosen for determining the probable distribution; however, if the maximum variations are as great as 100% of the arithmetic mean, it would be better, unless the data available were extremely numerous, to choose intervals of 10 per cent.

As a result of many studies of the accidental errors of observation (on which is based the curve representing the probability of errors), the following axioms, derived from experience, are stated:

1. Small errors occur more frequently, or are more probable, than large ones.
2. Positive and negative errors of the same magnitude are equally probable, and in a large number of observations are equally frequent.
3. Very large errors do not occur.

It is not necessarily true that the probable variations in other classes of happenings follow the laws of the probability of errors of observation. For example, if dots are placed at random on a straight line of definite length, the probability is equal that in equal lengths of the line there will be the same number of dots. This would be in violation of the first axiom just given; and it is only necessary for the happening which is being investigated to violate one of these axioms in order to vitiate (as far as that particular happening is concerned) any subsequent reasoning based on these axioms.

In order to make an accurate study of the probability of the occurrence of an event, the happening of which is governed by many complex influences, it would be necessary to have an infinite number of data. This, of course, is not practicable; therefore, it is the purpose of this paper to endeavor to determine the actual distribution of the variations in the yearly run-off of rivers, by a study of the measured run-off of thirty-five California streams.

The Data Used.

The data on the yearly run-off used in this paper were taken from the Water Supply Papers of the United States Geological Survey for all records up to September, 1914. The records given of the yearly run-off since September, 1914, were obtained from as yet unpublished manuscript furnished through the courtesy of the District Engineer of the San Francisco Office of the Water Resources Branch of the U. S. Geological Survey. These values of the yearly run-off for thirty-five California streams, which are the basic data used in this investigation, are given in Table 1.

The run-off of the rivers of California varies considerably from year to year, due to the semi-arid climate. Also, because of the mountainous topography, the rainfall and, consequently, the run-off is quite different for various portions of the drainage area of each stream. For the rivers flowing from the Sierra Nevada Mountains the average rainfall varies from 10 or 20 in. to 40 or 60 in. on each

TABLE 1.—YEARLY RUN-OFF OF THIRTY-FIVE CALIFORNIA STREAMS, IN PERCENTAGES OF THE ARITHMETIC MEANS OF THE TOTAL OBSERVATIONS ON EACH STREAM FOR SEASONAL YEARS, OCTOBER TO SEPTEMBER, INCLUSIVE.

Data compiled from Water Supply Papers Nos. 298, 299, 300, 329, 330, 331, 359, 360, 361, 389, 390, and 391, and from the yet unpublished records of the Water Resources Branch of the U. S. Geological Survey.

Seasonal Year.	(1) Sacramento River at Jellys Ferry, Cal., Oct., 1895-Dec., 1903. Drainage area, 10 200 sq. miles; and at Red Bluff, Cal., Jan., 1902-Oct., 1914. Drainage area, 10 400 sq. miles. Length of record, 19 years. Mean run-off, 9 865 000 acre-ft. C. V. = 0.29.	(2) Stony Creek, near Fruto, Cal., Oct., 1901-Sept., 1912. Drainage area, 601 sq. miles. Length of record, 11 years. Mean run-off, 548 700 acre-ft. C. V. = 0.40.	(3) North Fork of the Feather River below Prattville, Cal., Oct., 1905-Sept., 1914. Drainage area, 606 sq. miles. Length of record, 9 years. Mean run-off, 885 800 acre-ft. C. V. = 0.27.	(4) Feather River, at Oroville, Cal., Oct., 1902-Sept., 1914. Drainage area, 3 640 sq. miles. Length of record, 12 years. Mean run-off, 5 768 000 acre-ft. C. V. = 0.43.	(5) Butte Creek, near Butte Valley, Cal., Oct., 1905-Sept., 1914. Drainage area, 78 sq. miles. Length of record, 9 years. Mean run-off, 86 100 acre-ft. C. V. = 0.41.	(6) Yuba River, near Smartsville, Cal., Oct., 1903-Sept., 1914. Drainage area, 1 230 sq. miles. Length of record, 11 years. Mean run-off, 2 885 000 acre-ft. C. V. = 0.40.	(7) Bear River, at Van Trent, Cal., Oct., 1904-Sept., 1914. Drainage area, 293 sq. miles. Length of record, 10 years. Mean run-off, 382 600 acre-ft. C. V. = 0.57.	(8) American River, at Fairoaks, Cal., Oct., 1904-Sept., 1914. Drainage area, 1 910 sq. miles. Length of record, 10 years. Mean run-off, 3 513 000 acre-ft. C. V. = 0.54.	(9) Cache Creek, at Lower Lake, Cal., Oct., 1901-Sept., 1914. Drainage area, 500 sq. miles. Length of record, 13 years. Mean run-off, 349 500 acre-ft. C. V. = 0.58.	(10) Cache Creek, at Yolo, Cal., Oct., 1903-Sept., 1914. Drainage area, 1 230 sq. miles. Length of record, 11 years. Mean run-off, 623 300 acre-ft. C. V. = 0.69.	(11) Putah Creek, at Winters, Cal., Oct., 1905-Sept., 1914. Drainage area, 805 sq. miles. Length of record, 9 years. Mean run-off, 403 200 acre-ft. C. V. = 0.69.
1895-96	103.3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1896-97	103.3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1897-98	50.3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1898-99	58.8	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1899-00	86.4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1900-01	89.5	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1901-02	113.4	119.2	.....	.....	.....	.....	.....	.....	.....	.....	.....
1902-03	99.0	105.1	.....	.....	.....	.....	.....	.....	105.3	.....	.....
1903-04	161.2	137.2	.....	161.9	.....	142.0	.....	.....	80.1	.....	.....
1904-05	107.4	92.5	.....	77.9	.....	83.2	83.9	59.3*	162.7	142.9	.....
1905-06	112.5	97.5	111.3	116.5	129.0	125.8	146.8	135.8	97.8	100.9	.....
1906-07	138.9	139.3	147.1	162.0	148.7	158.1	189.9	162.9	109.1	118.8	125.9
1907-08	78.3	61.5	82.8	60.5	61.9	55.1	49.1	41.3	152.7	157.8	149.2
1908-09	135.9	162.8	118.0	128.1	112.3	134.4	137.2	129.3	58.7	49.6	42.9
1909-10	90.4	64.0	90.6	78.0	64.2	92.5	66.9	100.8	208.0	205.4	190.2
1910-11	100.6	97.5	118.1*	121.1	151.0*	121.7	133.8	184.7	52.4	40.1	49.2
1911-12	64.6	23.1	63.0	36.3	44.8	39.2	22.9	35.9	70.7	79.6	107.9*
1912-13	69.3	.....	70.3	46.3	64.9	48.5	27.7	40.7	14.2	5.9	12.5*
1913-14	136.8	.....	98.5	134.2	123.2	99.5	142.0	112.6	165.3	187.9	193.3

* Indicates yearly run-off partly estimated by the writer.

TABLE 1.—(*Continued*).

Seasonal Year.	(12) Kern River, at Bakersfield, Cal., Oct., 1893-Sept., 1914, Drainage area, 2 945 sq. miles. Length of record, 21 years. Mean run-off, 742 500 acre-ft. $C.\ V. = 0.56$.	(13) Tule River, near Porterville, Cal., Oct., 1901-Sept., 1914. Drainage area, 366 sq. miles. Length of record, 13 years. Mean run-off, 127 900 acre-ft. $C.\ V. = 0.69$.	(14) Kaweah River, near Three Rivers, Cal., Oct., 1903-Sept., 1914. Drainage area, 530 sq. miles. Length of record, 11 years. Mean run-off, 478 000 acre-ft. $C.\ V. = 0.57$.	(15) Kings River, near Sanger, Cal., Oct., 1893-Sept., 1914. Drainage area, 1 740 sq. miles. Length of record, 19 years. Mean run-off, 1 888 000 acre-ft. $C.\ V. = 0.42$.	(16) Merced River, at Merced Falls, Cal., Oct., 1901-Sept., 1913. Drainage area, 1 090 sq. miles. Length of record, 12 years. Mean run-off, 1 176 000 acre-ft. $C.\ V. = 0.54$.	(17) Tuolumne River and Canals, at La Grange, Cal., Oct., 1895-Sept., 1914. Drainage area, 1 600 sq. miles. Length of record, 19 years. Mean run-off, 2 100 000 acre-ft. $C.\ V. = 0.42$.	(18) Stanislaus River, at Knights Ferry, Cal., Oct., 1903-Sept., 1914. Drainage area, 935 sq. miles. Length of record, 11 years. Mean run-off, 1 538 000 acre-ft. $C.\ V. = 0.53$.	(19) Mokelumne River, at Electra, Cal., Oct., 1903-Dec., 1904. Drainage area, 537 sq. miles, and at Clements, Cal., Jan., 1905-Sept., 1917. Drainage area, 642 sq. miles. Length of record, 14 years. Mean run-off, 976 100 acre-ft. $C.\ V. = 0.42$.	(20) Cosumnes River, at Michigan Bar, Cal., Oct., 1907-Sept., 1917. Drainage area, 534 sq. miles. Length of record, 10 years. Mean run-off, 433 800 acre-ft. $C.\ V. = 0.56$.	(21) San Joaquin River, at Friant, Cal., Oct., 1907-Sept., 1917. Drainage area, 1 640 sq. miles. Length of record, 10 years. Mean run-off, 2 112 000 acre-ft. $C.\ V. = 0.48$.	(22) Truckee River, at Tahoe, Cal., Oct., 1900-Sept., 1916. Drainage area, 519 sq. miles. Length of record, 16 years. Mean run-off, 270 600 acre-ft. $C.\ V. = 0.58$.	(23) Truckee River, at Nev.-Cal. State Line, Oct., 1899-Aug., 1912. Drainage area, 925 sq. miles; and at Iceland, Cal., Sept., 1912-Sept., 1916. Drainage area, 944 sq. miles. Length of record, 17 years. Mean run-off, 769 500 acre-ft. $C.\ V. = 0.87$.
1893-94	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1894-95	70.5	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1895-96	137.3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1896-97	84.3	.....	.....	98.3	.....	75.7	.....	.....	.....	.....	.....	.....
1897-98	119.0	.....	.....	111.1	.....	116.2	.....	.....	.....	.....	.....	.....
1898-99	38.8	.....	.....	46.8	.....	45.7	.....	.....	.....	.....	.....	.....
1899-00	44.6	.....	.....	64.8	.....	63.3	.....	.....	.....	.....	.....	.....
1900-01	43.0	.....	.....	68.5	.....	77.6	.....	.....	.....	.....	.....	.....
1901-02	117.4	.....	.....	166.8	.....	129.5	.....	.....	.....	.....	.....	.....
1902-03	76.6	.....	.....	82.3	.....	76.7	.....	.....	.....	.....	.....	.....
1903-04	75.0	87.5	87.5	89.8	70.5	93.8	.....	.....	.....	.....	.....	.....
1904-05	61.1	54.9	78.0	92.4	75.1	126.7	131.1	.....	.....	.....	.....	.....
1905-06	73.8	53.5	70.7	75.9	93.6	81.9	55.0	135.2	.....	.....	.....	.....
1906-07	240.0	261.8	228.2	173.2	76.7	168.0	155.2	65.7	.....	.....	.....	.....
1907-08	128.9*	120.3	124.1	146.1	173.5	179.0	182.7	138.3	.....	.....	.....	.....
1908-09	66.0*	68.6	52.9	54.7	181.2	51.0	38.7	171.1	.....	.....	.....	.....
1909-10	219.0	222.6	167.3	149.2	135.9	126.2	123.3	49.2	.....	34.6*	54.9*	.....
1910-11	99.6	91.4	78.2	94.5	97.0	99.1	89.4	117.8	.....	147.1	137.3	35.8
1911-12	134.6	94.6	114.2	150.3	180.3	102.3	151.3	92.9	.....	106.8	96.6	47.0
1912-13	56.7	38.9	43.3	51.4	43.8	50.1	38.3	156.8	.....	202.2	108.7	57.7
1913-14	146.8	32.8	46.2	50.1	57.5	51.9*	32.2	40.3	.....	32.0	49.7	144.2
1914-15	.....	97.6	101.7	135.4	.....	124.7	103.1	48.3	.....	29.3	41.1	103.4
1915-16	.....	.....	.....	.....	.....	.....	.....	110.7	.....	126.3	135.9	62.5
1916-17	.....	.....	.....	.....	.....	.....	.....	84.3	.....	94.1	93.1	54.7
								105.5	.....	131.9	131.1	70.6
								88.9	.....	96.0	92.0	72.1
												104.8
												.....

* Indicates yearly run-off partly estimated by the writer.

* Indicates yearly run-off partly estimated by the writer.

TABLE 1.—(*Continued*).

Seasonal Year.	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)
	Donner Creek, at Truckee, Cal., Oct., 1902–Sept., 1913. Drainage area, 30 sq. miles. Length of record, 11 years. *Mean run-off*, 64 600 acre-ft. *C. V.* = 0.50.	West Fork of the Carson River, at Woodfords, Cal., Oct., 1900–Sept., 1915. Drainage area, 70 sq. miles. Length of record, 15 years. Mean run-off, 113 000 acre-ft. *C. V.* = 0.35.	Carson River, at Empire, Nev., Oct., 1900–Sept., 1917. Drainage area, 988 sq. miles. Length of record, 17 years. Mean run-off, 402 000 acre-ft. *C. V.* = 0.41.	Colorado River, at Yuma, Ariz., Oct., 1901–Sept., 1917. Drainage area, 225 000 sq. miles. Length of record, 16 years. Mean run-off, 17 170 000 acre-ft. *C. V.* = 0.32.	Owens River, at Round Valley, Cal., Oct., 1903–Sept., 1916. Drainage area, 450 sq. miles. Length of record, 13 years. Mean run-off, 202 400 acre-ft. *C. V.* = 0.21.	Rock Creek, near Round Valley, Cal., Oct., 1903–Sept., 1916. Drainage area, 46 sq. miles. Length of record, 13 years. Mean run-off, 32 000 acre-ft. *C. V.* = 0.36.	Pine Creek, near Round Valley, Cal., Oct., 1903–Sept., 1916. Drainage area, 32 sq. miles. Length of record, 13 years. Mean run-off, 20 900 acre-ft. *C. V.* = 0.46.	Santa Ana River and Canals, at Metone, Cal., Oct., 1901–Sept., 1917. Drainage area, 182 sq. miles. Length of record, 16 years. Mean run-off, 78 900 acre-ft. *C. V.* = 0.56.	San Gabriel River and Canals, at Azusa, Cal., Oct., 1895–Sept., 1917. Drainage area, 222 sq. miles. Length of record, 22 years. Mean run-off, 125 400 acre-ft. *C. V.* = 0.81.	Arroyo Seco, at Soledad, Cal., Oct., 1900–Sept., 1917. Drainage area, 215 sq. miles. Length of record, 17 years. Mean run-off, 159 700 acre-ft. *C. V.* = 0.58.	Coyote River, at Madrone, Cal., Oct., 1902–Sept., 1917. Drainage area, 188 sq. miles. Length of record, 10 years. Mean run-off, 87 300 acre-ft. *C. V.* = 0.76.	Millers Creek, at Lorella, Ore., Oct., 1904–Sept., 1914. Drainage area, 270 sq. miles. Length of record, 10 years. Mean run-off, 66 600 acre-ft. *C. V.* = 0.45.
1895–96	.....	.....	.....	.....	.....	.....	.....	.....	21.6	.....	.....	.....
1896–97	.....	.....	.....	.....	.....	.....	.....	.....	72.4	.....	.....	.....
1897–98	.....	.....	.....	.....	.....	.....	.....	.....	18.3	.....	.....	.....
1898–99	.....	.....	.....	.....	.....	.....	.....	.....	7.7	.....	.....	.....
1899–00	.....	.....	.....	.....	.....	.....	.....	.....	9.7	.....	.....	.....
1900–01	.....	.....	96.3	.....	.....	.....	.....	.....	76.7	112.8	.....	.....
1901–02	.....	62.9	65.9	46.0	.....	.....	.....	.....	19.0	62.6	.....	.....
1902–03	.....	88.4	76.4	64.6	.....	.....	.....	93.6*	84.5	65.8	.....	.....
1903–04	79.3	75.8	124.6	57.5	.....	80.6	107.7	75.5	82.9	87.2	96.4	.....
1904–05	178.0	113.3	62.6	110.1	90.9	67.5	69.9	38.8	67.5	73.2	41.0	.....
1905–06	72.9	73.2	134.9	111.8	82.5	134.3	153.5	98.1	127.6	128.4	36.4	50.4
1906–07	149.2	146.4	182.3*	151.3	122.0	196.9	157.8	183.1	188.0	191.7	134.0	121.9
1907–08	156.3	187.4	51.2*	79.2	93.4	80.3	60.7	396.6	278.8	288.6	288.6	180.1
1908–09	58.8	64.6	108.3	151.8	105.2	98.5	78.8	78.8	61.8	43.4	54.1	30.9
1909–10	94.2	92.0	88.6	87.4	98.9	90.6	143.9	101.8	143.4	149.1	201.6	100.3
1910–11	110.3	133.9	155.6	94.3	124.9	137.9	101.9	73.5	110.8	53.0	51.9	108.5
1911–12	134.4	65.1	44.8	114.1	77.1	76.6	102.1	121.9	217.6	182.2	144.3	129.3
1912–13	44.4*	48.0	69.9	69.9	70.6	71.0	63.5	58.7	61.5	23.2	7.3	70.5*
1913–14	21.5	66.0	145.1	115.9	122.0	121.8	122.0	47.6	40.1	6.9	.....	78.0*
1914–15	.....	96.5	115.9	122.0	88.5	126.2	126.2	88.9	235.8	163.5	.....	121.1*
1915–16	.....	77.6	80.5*	92.0	88.5	87.5	66.5	138.0	115.1	130.9	.....	.....
1916–17	.....	.....	117.6	125.2	92.4	117.2	75.6	232.4*	222.2	102.2	.....	.....
	.....	.....	117.3	128.7	.....	.....	.....	92.1	73.3	113.8	.....	.....

* Indicates yearly run-off partly estimated by writer.

drainage area; the low precipitation occurring on the valley and foot-hill area, while the larger quantities prevail in the higher mountain regions.

The precipitation throughout California occurs almost entirely during the winter months, and in the foot-hill area it is mostly in the form of rain, while the run-off from this area occurs for the most part during the season of rainfall. However, in the high Sierra the precipitation is chiefly in the form of snow, which melts slowly and sustains the flow of the rivers draining these higher altitudes well into the summer months. All the streams are at their lowest stage during July, August, and September.

The Coast Range streams are similar to those draining the foot-hill area of the Sierra Nevada Mountains, in that they depend entirely on the rainfall of the winter months for their run-off, and are not snow-fed.

Of the streams used in the investigation, twenty-five drain the Sierra Nevada; nine are Coast Range streams; and one—the Colorado River—is only a California stream in that, for a portion of its course, it forms the boundary between California and Arizona. The streams on the eastern slope of the Sierra Nevada Mountains drain into the area known as the Great Basin, each flowing into some lake where the annual influx of water is lost through evaporation into the desert air. The streams on the western slope of the Sierra Nevada Mountains drain either into the Sacramento or the San Joaquin Valley, where they find their way to the ocean through San Francisco Bay, as do those from the eastern slopes of the Coast Range. The streams draining the western slopes of the Coast Range flow directly into the Pacific Ocean. The Colorado River receives most of its run-off from the Rocky Mountains of Colorado and Wyoming, and empties into the Gulf of California.

In Table 1, the streams are numbered consecutively and will be referred to hereafter, for the most part, by these numbers. For each stream the following information is given: The name of the stream, the location of the gauging station, the years during which the record was taken, the drainage area above the gauging station, in square miles, the length of the record, in years, the mean yearly run-off, and the coefficient of variation, *C. V.** Then follows the measured yearly run-off for each year as a percentage of the arithmetic mean of the total observations on each stream.

It will be noted, by referring to Table 1, that records for certain years are marked as being partly estimated by the writer. This was done in the case of some streams in order to fill out the record. The estimates for the missing months were made in various ways—by comparison with neighboring streams, or with other gauging stations on the same streams, or by using the average flow recorded in other years for the missing month. The months missing were usually either October, November, and December, or June, July, and August. As the streams are always at a low stage during these periods, a considerable error in estimating the run-off for any one or two of these months would not have a very great influence on the total run-off for the year.

* The coefficient of variation indicates the relative dispersion of the data on the individual streams, and is found by the formula, $C.\ V. = \frac{\sqrt{\Sigma v^2}}{\sqrt{n-1}}$, where $C.\ V.$ = coefficient of variation, v = percentage of variation of each year from the mean, and n = number of years in the record.

Accuracy of the Data.

In making a study of this nature, the normal variations of the yearly run-off are complicated still further by inaccuracies in the measurements of the quantities involved. However, it was believed that, in order to get positive results, quantity was of more importance than quality, so the records of some streams were included of which, due to shifting of beds, torrential character of the stream at high stages, or other reasons, it was stated in the Water Supply Papers of the U. S. Geological Survey that the record was poor. Another cause for possible inaccuracies in the measurement of the yearly run-off, may be ascribed to the practice of reading the gauges only once or, in some cases, twice a day. Automatic water-stage recorders which have been installed in recent years at some of the more important gauging stations, have shown the error of this method in the case of streams having their sources in the high Sierra, and which are subject to large diurnal fluctuations due to the varying rate of the melting snow during different hours of the day. An example of the inaccuracies of a single daily observation of the gauge height can be cited for the Feather River, at Oroville, Cal. (Stream No. 4 in Table 1), which is noted on pages 313-315 of Water Supply Paper No. 361 of the U. S. Geological Survey. The United States Weather Bureau originally maintained a vertical staff gauge in two sections on a pier of the highway bridge at Oroville near the right bank of the river. In November, 1912, a Friez water-stage recorder was installed on the right bank, 200 ft. below the bridge, at the same datum as the old staff gauge, and the observations from this recorder show that the minimum stage for the day occurs about 7 A. M., the time at which the Weather Bureau observations were recorded. It is evident that the calculations of the run-off of the Feather River at this gauging station, which are based on the Weather Bureau observations, are in error to a considerable degree.

It is highly possible that in the case of the records of this stream, as well as for some of the other streams, the record of each individual year may be in error by as much as 25 per cent. It was hoped by including many streams that, in the long run, these accidental errors in measurement would compensate each other.

Method of Using Data.

It is evident, of course, that the best way to investigate the variations of the yearly stream flow would be to treat each stream by itself. Some attempt was made to apply this method, but it had to be abandoned, for the reason that insufficient data exist on any one stream. To make any intelligent study of the probable variations of the yearly run-off for each river by itself, it would be necessary to have a record of at least 100 years. No such record exists on any stream in the United States; while for the California rivers, the longest record available is the 22-year record of the San Gabriel River and Canals at Azusa, from October, 1895, to September, 1917. (Stream No. 32 in Table 1). As some other method of studying the probabilities had to be adopted, it was decided to combine the records on the thirty-five streams, and to treat the yearly run-offs only in terms of the percentages to the arithmetic means of the total observations

on each stream, this last being done to reduce the measurements on the separate streams to some common base.

Perhaps there may be some doubt as to whether the probable variations from the arithmetic mean of all these streams taken collectively, would be the same as for any one stream individually; however, the results obtained by this method would appear to justify its use.

The Probability Curves.

The records for the thirty-five California streams, comprising 470 stream-years, or an average of ($570 \div 35 =$) 13.4 years on each stream, were used in making Probability Curve No. 1, shown on Fig. 1, in the lower part of which, the measured yearly run-offs for each stream are plotted as percentages of the arithmetic means, each stream occupying a line by itself, and are referred to by the serial numbers given in Table 1.

In order to determine the distribution of these plotted observations, counts were made of the actual number of points which had occurred within successive intervals of 10% of the arithmetic mean. These counts were made for two series of 10% intervals (in each case using the arithmetic mean as 100%), as follows:

First.—Beginning at zero and considering the intervals of 0-10, 10-20, 20-30, * * *, 90-100, 100-110, * * *, 260-270 and 270-280 per cent.

Second.—Beginning at 5% and considering the intervals of 5-15, 15-25, 25-35, * * *, 95-105, 105-115, * * *, 265-275, and 275-285 per cent.

The result of the count for the first series of 10% intervals is shown in Table 2, in which the actual number of yearly run-offs which had occurred within each of these 10% intervals are also expressed as percentages of the total number of stream-years (or of 470). A tabulation similar to that in Table 2 was made, considering the intervals of 5-15, 15-25, 25-35, * * * 95-105, 105-115, * * * 265-275, and 275-285 per cent. These percentages for both series are plotted in the upper part of Fig. 1. The points so plotted arrange themselves into a curve somewhat similar to the curve of the probability of errors; but, on the other hand, differing from it markedly in some respects. In the first place, it will be noted that the curve is not symmetrical with respect to the arithmetic mean or to any other point, the variations exceeding the arithmetic mean extending, on the right of the 100% line, much farther than those less than the mean, on the left of that line. In the second place, the summit of the curve is located at about 80% of the arithmetic mean; the actual maximum percentage of observations occurring in the 10% intervals (for which the counts were made) was, for the first series, at the interval between 70% and 80% of the arithmetic mean, where 9.79% of the total number of stream-years was recorded; and for the second series at the intervals between 75% and 85% of the arithmetic mean where 8.94% of the total number of stream-years was noted. This indicates that the most probable value of the yearly run-off is less than the arithmetic mean, for, as stated in the discussion of the principles of probability, the most probable occurrence is that which has the greatest mathematical certainty, or which occurs with greater frequency than any other.

TABLE. 2.—Distribution of Yearly Run-off of 470 Stream-Years for the Combined Records of Thirty-five California Streams, Reduced to Percentages of the Arithmetic Means of the Total Observations on Each Stream.

(1) Intervals of 10% of arithmetic mean.	(2) Number of observations occurring in intervals.	(3) Percentage of number of observations in Column (2) to total, or to 470.	(4) Cumulative percentages of Column (3).
0– 10	5	1.06	1.06
10– 20	5	1.06	2.12
20– 30	11	2.34	4.46
30– 40	18	3.83	8.29
40– 50	34	7.24	15.53
50– 60	31	6.60	22.13
60– 70	36	7.66	29.79
70– 80	46	9.79	39.58
80– 90	28	5.96	46.54
90–100	43	9.14	54.68
100–110	26	5.53	60.21
110–120	30	6.39	66.60
120–130	36	7.66	74.26
130–140	33	7.02	81.28
140–150	22	4.68	85.96
150–160	14	2.98	88.94
160–170	15	3.19	92.13
170–180	5	1.06	93.19
180–190	12	2.55	95.74
190–200	3	0.64	96.38
200–210	5	1.06	97.44
210–220	2	0.43	97.87
220–230	3	0.64	98.51
230–240	3	0.64	99.15
240–250	2	0.43	99.58
250–260	0	0.00	99.58
260–270	1	0.21	99.79
270–280	1	0.21	100.00
280–290	0	0.00	100.00
290–300	0	0.00	100.00
Total	470	100.00	.....

Both these differences between Probability Curve No. 1 (showing the distribution of the yearly run-off with respect to the arithmetic mean) and the curve representing the probable distribution of errors, are in contradiction of the latter. As is known, the curve of the probability of errors is symmetrical with respect to the arithmetic mean, and also the summit of this curve or the most probable value is at the arithmetic mean. In fact, the whole theory of Least Squares is based on the assumption of the arithmetic mean as representing the most probable value of several independent measurements of a quantity.

It was thought necessary to investigate whether the inclusion of streams having large variations from the arithmetic mean did not perhaps alter the shape of the probability curve from the form it would assume if only the streams having more regular regimens were considered in determining the probable variations.

For this reason, Probability Curve No. 2 shown on Fig. 2 was constructed. The streams used in forming this curve were chosen as having no minimum measured yearly run-off which was less than 35% of their respective arithmetic means. The only particular reason in choosing 35% of the arithmetic mean as the minimum permissible variation in this second tabulation, was that by so doing approximately one-half of the total 470 stream-years were eliminated. By omit-

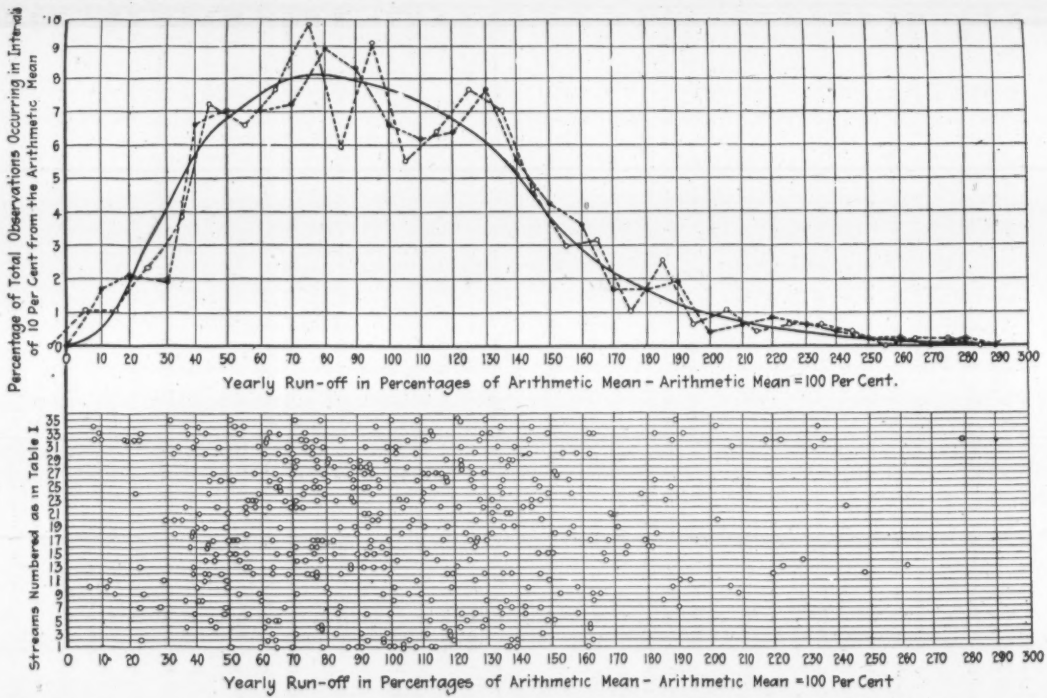

FIG. 1.—PROBABILITY CURVE NO. 1. SHOWING DISTRIBUTION OF YEARLY RUN-OFF WITH RESPECT TO THE ARITHMETIC MEAN FOR THE COMBINED RECORDS OF THIRTY-FIVE CALIFORNIA STREAMS.

The circles in the upper part of this diagram represent the percentage of the total number of observations (or of 470 stream-years) occurring in intervals of 10% from the arithmetic mean. Open circles represent the percentage of observations occurring in intervals 0-10, 10-20, etc. Black circles represent the percentage of observations occurring in intervals 5-15, 15-25, etc. The smooth curve represents the ideal probability curve assumed for constructing the cumulative or integral curve No. 3 on Fig. 3. The circles on the lower part of the diagram show graphically the measured yearly stream flows expressed as percentages of the arithmetic means of the total observations on each stream, as computed in Table 1.

ting Streams Nos. 2, 7, 9, 10, 11, 13, 18, 20, 24, 30, 31, 32, 33, 34, and 35, the number of streams used is reduced to 20, and the number of stream-years to 283, or an average of 14.2 years on each stream.

The data for these 20 streams were plotted on Fig. 2, and counts of the number of stream-years occurring in 10% intervals were made in exactly the same manner as previously explained for Probability Curve No. 1. The results of these counts are shown plotted on the upper part of Fig. 2.

The maximum percentages of the total number of measured stream-years, which were recorded in the chosen 10% intervals, occurred in the same intervals as in the previous tabulation of the entire 35 streams, namely, between 70% and 80% of the arithmetic mean for the first count, and between 75% and 85% of the arithmetic mean for the second count. However, in this case, the maximum percentages were 11.32% and 11.29%, respectively, of the total number of 283 stream-years, as compared with 9.79% and 8.94% of the 470 stream-years recorded for the corresponding 10% intervals of the arithmetic mean in the former tabulation.

It will be noted that the two probability curves, No. 1 and No. 2, are similar in every respect, in that both show unsymmetrical probabilities and a most probable value of the yearly run-off which is less than the arithmetic mean. The two curves differ only in the extent of the variations from the arithmetic mean and in the heights of the ordinates under their summits.

Discussion of the Probability Curves.

The fact that the probability curves representing the variations of the yearly stream flow are unsymmetrical is in violation of the second axiom derived by other writers as the result of experience in studying the facility of errors in observations. This axiom, already stated, requires the positive and negative errors from the arithmetic mean of the same magnitude to be equally frequent and in the long run equally liable to occur, and the fulfillment of this axiom requires a symmetrical curve.

In order to make certain that a few erroneous observed measurements did not account for the unsymmetricalness of the probability curves shown in Figs. 1 and 2, a study was made of the relation between the minimum and maximum observed yearly run-off for each of the thirty-five streams tabulated in Table 1. This study was made by taking, for each individual stream shown in Table 1, the minimum and the maximum observed yearly stream flow, and the percentage variations of these values from their respective means were then used to find the ratio of the maximum variation above the arithmetic mean to the maximum variation below the arithmetic mean. Evidently, a value of this ratio greater than unity indicates that the variation above the arithmetic mean is the greater, and *vice versa*.

The results of this investigation disclosed the fact that, of the thirty-five streams, there are only six cases where this ratio is less than unity to twenty-nine cases where it is greater than unity; in other words, there are twenty-nine cases where the maximum variation above the arithmetic mean is greater than that below, against only six cases where the reverse is true. The minimum value of the ratio is 0.82 for Stony Creek, at Fruto, Cal., and the maximum value is 2.47 for the

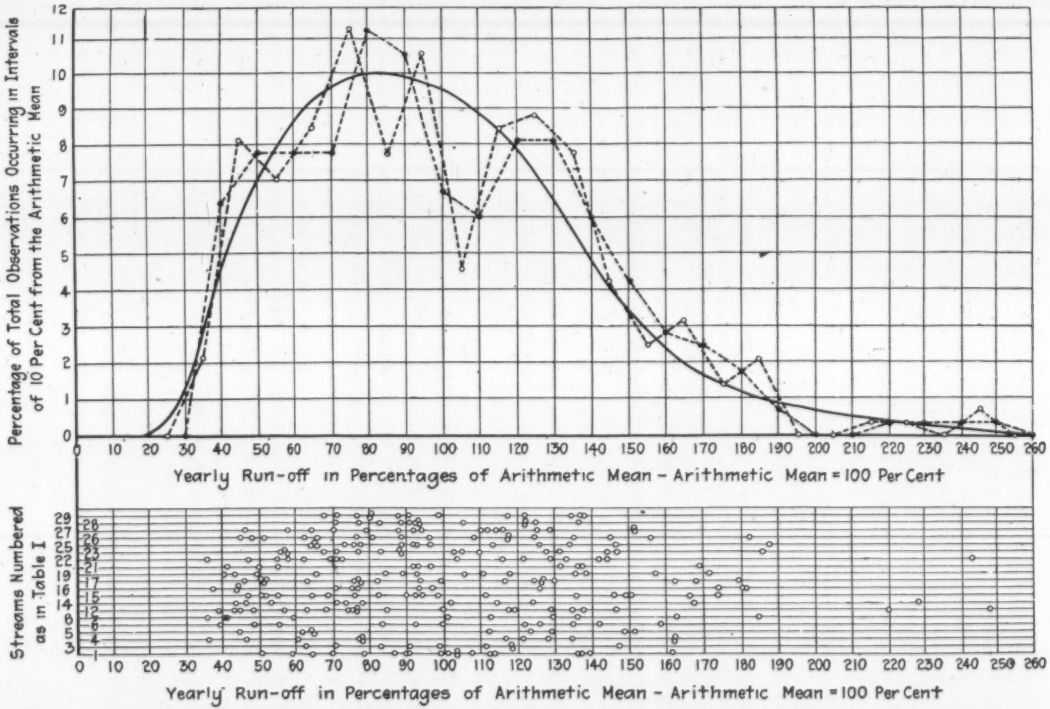

FIG. 2.—PROBABILITY CURVE NO. 2. SHOWING DISTRIBUTION OF YEARLY RUN-OFF WITH RESPECT TO THE ARITHMETIC MEAN FOR THE COMBINED RECORDS OF TWENTY CALIFORNIA STREAMS.

The circles in the upper part of the diagram represent the percentage of the total number of observations (or of 283 stream-years) occurring in intervals of 10% from the arithmetic mean. Open circles represent the percentage of observations occurring in intervals 30-40, 40-50, etc. Black circles represent the percentage of observations occurring in intervals 35-45, 45-55, etc. The smooth curve represents the ideal probability curve assumed for constructing the cumulative or integral curve No. 4 on Fig. 3. The circles on the lower part of the diagram show graphically the measured yearly stream flows expressed as percentages of the arithmetic means of the total observations on each stream, as computed in Table 1.

West Fork of the Carson River, at Woodfords, Cal., while the average value for all streams is 1.40.

Taking into consideration the inaccuracies known to exist in the measured quantities, it is believed that these results establish conclusively the unsymmetricalness of the probable variations of the yearly stream flow.

The second point brought out through the construction of the probability curves, that is, the fact that the most probable value is less than the arithmetic mean, also requires further discussion. The use of the arithmetic mean, as representing the most probable value of several independent measurements of any quantity, is considered by many as axiomatic. Where such observations are measurements of the same quantity, it has been definitely proven by many investigators that such is the case. However, it must be realized that the measurements of the annual run-off of a stream in different years, or for that matter the annual measurements of any natural phenomena, are not measurements of the same thing in the same sense that several measurements of the length of a geodetic base line, or of the angle between two fixed stars, are measurements of the same thing. In the latter case, there is only one real value of the quantity involved, and variations from this are due solely to the accidental errors of observation; but, in the former case, each measurement is an actual and independent observation, and variations between these quantities are due to accidents of a nature which are beyond the control of the observer.

The use of the arithmetic mean as the most probable value requires the compliance of the data with the three axioms on which the theory of Least Squares is based. The requirement that the distribution of the probabilities must be symmetrical is referred to by Mansfield Merriman, M. Am. Soc. C. E., in two places in his book.* On page 174, after discussing the application of the law of the probability of error to social and political statistics, it is stated that "the individuals of each type are clustered around its mean, according to the law of probability; and the several types are clustered around a general mean, according to the same law. This is true for all statistical data in which equal positive and negative deviations from the mean are equally probable; in other cases an unsymmetric distribution may occur." Again, on page 210, in discussing the use of the median in place of the arithmetic mean, it is stated that "the use of the median * * * does not seem warranted in cases of symmetric probability." It is seen from these references that stress is laid on the necessity of the symmetrical distribution of the data, in order that the law of the probability of errors may be applicable.

It is believed that these two examples give foundation and support to the assumption of the most probable value as less than the arithmetic mean for the case of the annual run-off of streams, and verifies the correctness of the results derived from the construction of Probability Curves Nos. 1 and 2.†

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* "Method of Least Squares", 1915 Edition.

† A verification of the conclusions reached as a result of this study has been recently found by the writer in the book, "Introduction to the Theory of Statistics," by G. Udny Yule. Mr. Yule states that the unsymmetrical curve is the most common type of probability curve, illustrations occurring in statistics from almost every source. A mathematical treatment of the type of probability (also called frequency) curves shown on Figs. 1 and 2 may be found in the book "Frequency Curves and Correlation," by W. Palin Elderton.

The Cumulative or Integral Probability Curves.

Having derived the probability curves, it was necessary to obtain the cumulative progressive percentages or the approximate integral of the area under such curves. This was first done by adding together cumulatively the actual percentages of the total number of observations occurring in the intervals of 10% from the arithmetic means. These cumulative percentages can be found in Column 4 of Table 2 for the results from Probability Curve No. 1 for the first series of 10% intervals; the tabulation of the results of the other series have been omitted, but are shown on Fig. 3.

In order to form a smooth curve through the points plotted on Fig. 3, the curves representing the assumed ideal probable distribution of the yearly run-off were drawn on Figs. 1 and 2. The curves so drawn for the first trial were not precisely the same as those shown on these diagrams, as these curves have been modified to make the two curves similar. The cumulative sums of the percentages of the total observations occurring in intervals of 10% from the arithmetic mean were found by reading the ordinates to the assumed ideal probability curves on Figs. 1 and 2, and these results were plotted as the cumulative or integral probability curves, Nos. 3 and 4, on Fig. 3, Curve No. 3 representing results from Curve No. 1, and Curve No. 4 representing results from Curve No. 2. As may be seen, these curves (of an ogee shape) have very similar characteristics, and an effort was made to adjust them, in order to make them exactly identical. In this and in the work of constructing the probability scale finally adopted, graphical methods were used throughout, and no attempt was made to analyze the curves by intricate mathematical calculations.

The cumulative or integral curves, Nos. 3 and 4, were found to intersect approximately on the line representing 50% of the total observations, and the exact point of intersection on the horizontal scale was finally selected, after one or two trials, at 94% of the arithmetic mean; in other words, the median value was assumed from a study of the data under consideration as being at 94% of the arithmetic mean. Also, the most probable value of the yearly stream flow (from the results shown by the probability curves) was taken at a point at which the sum of the percentages (vertical ordinates) under the curves equals (beginning at the end of the curve representing minimum stream flows) 37.5% of the total number of observations (or stream-years); or, the most probable value of the yearly stream flow for any particular stream, which will at least closely approximate the one previously defined, may be obtained by rejecting the maximum measured yearly stream flows to an amount equal to 25% of the total observations, and taking the arithmetic mean of the remaining 75 per cent.

As an initial step to establishing a relationship between Curves Nos. 3 and 4, the ratio was computed between the maximum variation above the median and the maximum variation below the median (assumed as stated before to be at 94% of the arithmetic mean). The maximum value of the yearly run-off, possible to occur from the results shown in Curve No. 3, was assumed as 300%; while the minimum value, taken from the same curve, was assumed as 0% (the actual observed maximum and minimum variations from the arithmetic mean are 278.8 and 5.9). These assumed maxima and minima give percentage variations from

the median of $(300 - 94 =)$ 206%, maximum variation above the median, and $(94 - 0 =)$ 94%, maximum variation below the median. The ratio of these two, therefore, would be $\frac{206}{94} = 2.19$. In order to keep this ratio the same for Curve No. 4, it was necessary to assume maximum and minimum probable yearly run-offs of 259.5 and 18.5, respectively. Then,

$$\frac{259.5 - 94}{94 - 18.5} = \frac{165.5}{75.5} = 2.19.$$

It may be noticed that the value of the ratio just found is different from the mean value of the ratio of the maximum variations above and below the arithmetic mean, as given on page 204. The ratio as computed there was 1.40, compared with 2.19 just determined. However, since the records on each stream were for a definite period of years, the value of 1.40 was not the ratio between absolute maximum variations; and, moreover, was computed with respect to the arithmetic mean rather than the median. The average length of the records on each stream was 13.42 years, or each year's run-off must be considered to represent the average happening for $(100 \div 13.42 =)$ 7.44% of the time. The values from Curve No. 3 at $\left(\frac{7.44}{2} =\right)$ 3.72% and $\left(100 - \frac{7.44}{2} =\right)$ 96.28% of the cumulative percentages of the total number of observations, are found to be 28% and 200% of the arithmetic mean. These percentages will give for the ratio, computed with respect to the median, a value of

$$\frac{200 - 94}{94 - 28} = \frac{106}{66} = 1.605;$$

or computed with respect to the arithmetic mean, a value of

$$\frac{200 - 100}{100 - 28} = \frac{100}{72} = 1.39.$$

This last value is comparable with that computed from the individual records, and checks it very closely.

To return from this digression: Two straight lines now were drawn on Fig. 3 to connect the points previously found; one line, *a b c*, through the points (0, 0) and (100, 300), and the other, *d b e*, through the points (0, 18.5) and (100, 259.5). These lines intersect at *b*, which is naturally on the ordinate through 94% of the arithmetic mean.

By referring each of the integral curves to these lines, they can be made exactly identical. The method of doing this is shown for two or three cases. For example, starting at Point *f* on Curve No. 3, where it intersects the abscissa through 10% of the total number of observations, this point is projected vertically to *g* on the straight line, *a b c; g*, then, is projected horizontally to *h* on the other straight line, *d b e; h*, in turn, is projected vertically downward to *j* on the 10% abscissa, which point should be and is where Curve No. 4 crosses this abscissa. Other examples are shown at the abscissas through 70% and 95% of the total number of observations. In the first case, starting at Point *k* on Curve No. 3 and following around *k l m n*, Point *n* on Curve No. 4 is reached; in the second case, the transition from one curve to the other is shown by Points *o p q r*. The

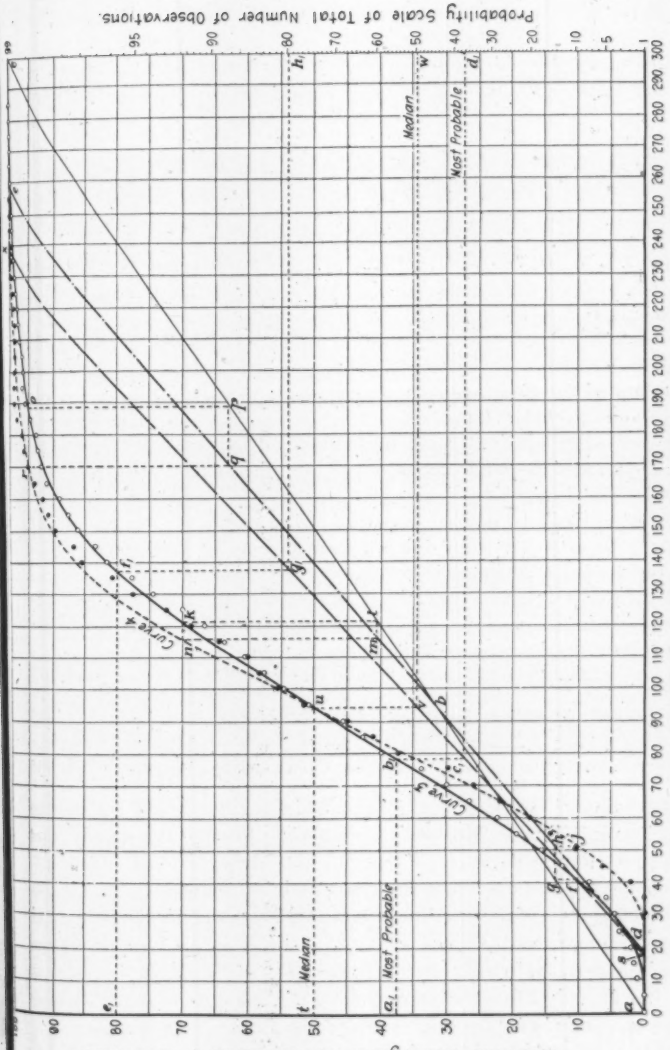

FIG. 3.—CUMULATIVE OR INTEGRAL PROBABILITY CURVES NOS. 3 AND 4, SHOWING CUMULATIVE DISTRIBUTION OF YEARLY RUN-OFF WITH RESPECT TO THE ARITHMETIC MEAN FOR THIRTY-FIVE AND TWENTY CALIFORNIA STREAMS TAKEN FROM FIGS. 1 AND 2, RESPECTIVELY.

Open circles indicate the cumulative percentages of the total number of actual observed yearly stream flows occurring in intervals of 10% from the arithmetic mean, and are derived from the results shown on Fig. 1. Black circles indicate similar cumulative percentages for the results shown on Fig. 2. The cumulative of integral curve No. 3 is derived from the assumed ideal probability curve of the distribution of the yearly run-off shown on Fig. 1 by adding cumulatively the percentages occurring under the curve between small, equally spaced intervals. The cumulative curve No. 4 is obtained in the same manner from the ideal probability curve shown on Fig. 2.

steps outlined can be reversed if desired, or can be applied to any point on either curve; in other words, starting at any point on either curve at an abscissa or line representing the scale of the total number of observations, and following through the steps outlined, a point on the other curve, and on the same abscissa, will be reached.

As previously stated, the curves shown on Figs. 1, 2, and 3 were arrived at only after several trials, and any change made in any one of them necessitated similar changes in all the others; therefore, the curves shown were finally adopted as those best representing the probable variations of the yearly stream flow that could be drawn, using the data which have been considered in this investigation.

The Probability Scale.

It would appear reasonable to consider that the probable variations in the yearly run-offs of any stream can be best represented by a curve similar to Curves Nos. 3 and 4 on Fig. 3. It is evident that by drawing other straight lines on Fig. 3 through *b*, the intersection of *a b c* and *d b e*, any number of integral curves could be constructed similar to the two already shown. Such procedure would be very awkward, if attempted for every stream to be studied; therefore, the probability scale of the Hydraulic Probability Paper was developed, so that (by using this new scale) the curves shown on Fig. 3 could be represented by straight lines. This scale is shown along the right-hand edge of Fig. 3, and the method used in obtaining it will now be explained.

In the first place, it will be noticed that this scale extends only from 1 to 99, or, in other words, that 1% has been dropped from each end of the curve. This was done for the reason that the extreme ends of the probability and integral curves are difficult to determine exactly, since only a limited amount of data was used in this investigation, rather than the infinite quantity theoretically necessary. For example, a certain maximum yearly run-off on any chosen stream might not be exceeded when the number of stream-years available was 100, 500, or 1 000; the probability of its re-occurrence would be for each case, respectively, $\frac{1}{100}$, $\frac{1}{500}$, and $\frac{1}{1\,000}$; or, in general, the greater the number of stream-years observed which did not exceed this maximum excessive yearly run-off, the smaller would become the probability that it ever will be so exceeded. Since only a total of 470 stream-years was used in the present investigation, it was thought advisable to eliminate 1% from each end of the integral curve and limit the study of the probable variations of the yearly run-off to that of the maximum exceeded only once in 100 years, and to that of the minimum exceeded not more than 99 times in 100 years. As the longest record of the measured yearly stream flows of any of the thirty-five streams given in Table 1 is for only 22 years, the projection of such data to cover the possible happenings in a period of 100 years, appears to be all that is warranted.

The probability scale could be obtained from either Curve No. 3 or Curve No. 4, but as the results would be identical in both cases, the method is shown on Fig. 3 as applied only to Curve No. 3. Therefore, the points where Curve No. 3 crossed the 1% and the 99% line on the vertical scale or scale of total number of observa-

tions, were noted. These occurred at 18% and 239% on the horizontal scale or scale of Percentages of Arithmetic Mean.

The probability scale could be made any length desired. In this case, a scale 10 in. long was chosen as being most convenient; hence, the points where Curve No. 3 crossed 1% and 99% on the vertical scale representing an amplitude of 98% were replotted at their proper places on the horizontal scale and 10 in. apart on the vertical scale. The points thus plotted were connected by the straight line, $s\ x$ (Fig. 3). It was then desired to make this line, $s\ x$, represent Curve No. 3 between the limits of 1% and 99% of the total number of observations, by deriving such a varying vertical scale on Fig. 3 (the scale showing the cumulative percentages of the total number of observations) that the straight line, when read by the new vertical scale in conjunction with the old horizontal scale (percentages of the arithmetic mean), would represent Curve No. 3 correctly and accurately.

Any curve can be represented by a straight line by the selection of the proper types of scales. The most common example of this is the representation of curves of general function, $y = a\ x^n$, by straight lines, when these functions are plotted on logarithmic cross-section paper.

By projecting vertically points on Curve No. 3 to the straight line, $s\ x$, and marking these points on this line with the same percentages as occurred on the vertical scale opposite the curve, a scale will be developed which will accomplish the result desired.

A more detailed explanation of the procedure described will now be given by means of a few examples. Starting at t, the 50% point on the vertical scale at the left-hand side of Fig 3, proceed horizontally to the right to u on Curve No. 3, from u continue vertically downward to v on the line, $s\ x$, Point v projected horizontally to the right will reach w, which will be the 50% point on the new variable or probability scale.

The transfer of 37½%, or the location of the most probable value, from the old to the new scale is shown by Points $a_1\ b_1\ c_1\ d_1$; and of the 80% point by Points $e_1 f_1 g_1 h_1$. By repeating this operation for each individual percentage point between 1% and 99%, on the arithmetic scale of the left-hand edge of Fig. 3, the whole probability scale shown along the right-hand edge of this diagram will have been developed.

After the probability scale had been derived by the method described, it was found necessary to smooth out inequalities in order to make the spacing between the whole percentage points increase uniformly from 37½%, the most probable value or summit of the probability curves. As shown on Fig. 3, with the spacings of the probability scale as finally adopted, all inequalities have been eliminated.

The Hydraulic Probability Paper.

The cross-section paper, which has been named by the writer, Hydraulic Probability Paper, is shown on Fig. 4. This paper is formed by a combination of the probability scale, derived in the manner just explained, with an ordinary or arithmetic scale. As shown on Fig. 4, the 10-in. vertical probability scale of Fig. 3 was used unchanged in length as the horizontal scale of the Probability Paper; while the vertical scale used for the latter (20 divisions to the inch) was adopted chiefly because thus 300% may be comprised within 7½ in., and thereby

all the varying percentage values encountered (of the yearly stream flows of each stream, expressed as percentages of its mean yearly stream flow) are fully covered.

To use the Hydraulic Probability Paper in investigating the variations in the yearly run-off of a stream, the data to be studied must first be arranged in order of magnitude, ranging from the minimum to the maximum values of the measured yearly run-off. If measurements of the run-off have been made for m stream-years, each such yearly value will represent (so far as the available data of past occurrences can represent correctly the future) what will be the probable yearly stream flow for $\frac{1}{m}$th of the future years. The horizontal or probability scale then is divided into m equal parts numerically, and the run-off values for each year are plotted in their order of magnitude, each value in the middle of its corresponding percentage space and opposite its proper place on the vertical scale.

To give a numerical illustration: If there was a measured run-off of 10 years on a certain stream, the probability scale would be divided into 10 equal spaces of 10% each. The minimum observed run-off would be plotted at 5%, the next largest run-off at 15%, the next at 25%, and so on, up to the maximum value which would be plotted at 95 per cent. The location on the probability scale at which to plot the nth term of a series of m observations of the yearly run-off, can be found by the formula:

$$P = \frac{2n - 1}{2m}$$

where P = the proper percentage on the probability scale;
m = the total number of measured yearly stream flows;
n = the order of magnitude of the stream flow of the year to be plotted.

After a record has been plotted, a straight line drawn through or close to the plotted points represents the probable variations in the yearly run-off of the stream being studied.

Since the spacing of the Probability Paper extends only between 99% and 1%, those points mark, respectively, the maximum value of yearly stream flow which probably will be exceeded only 1% of the time, or not more than once in 100 years, and a minimum yearly stream flow than which a smaller value probably will not occur oftener than once in 100 years. Similarly, any point on the probability scale represents the percentage of time or number of years in 100 that the yearly run-off will be equal to or less than a specified value. This scale, therefore, can be used to denote relative wetness, since absolute wetness or the maximum yearly stream flow which would probably never be exceeded would then be 100. By taking the difference between 100 and the scale readings, the percentage of time or number of years in 100 that the run-off will equal or exceed a specified value will be denoted. These differences between 100 and the scale readings can be used to indicate relative dryness, since now the minimum yearly stream flow (that is probable ever to occur) which would be zero in relative wetness would become 100 in relative dryness.

The vertical or arithmetic scale is used to represent the yearly stream flow which can be expressed as percentages of the arithmetic mean or in any convenient unit terms either for the whole drainage area or per square mile of drainage area,

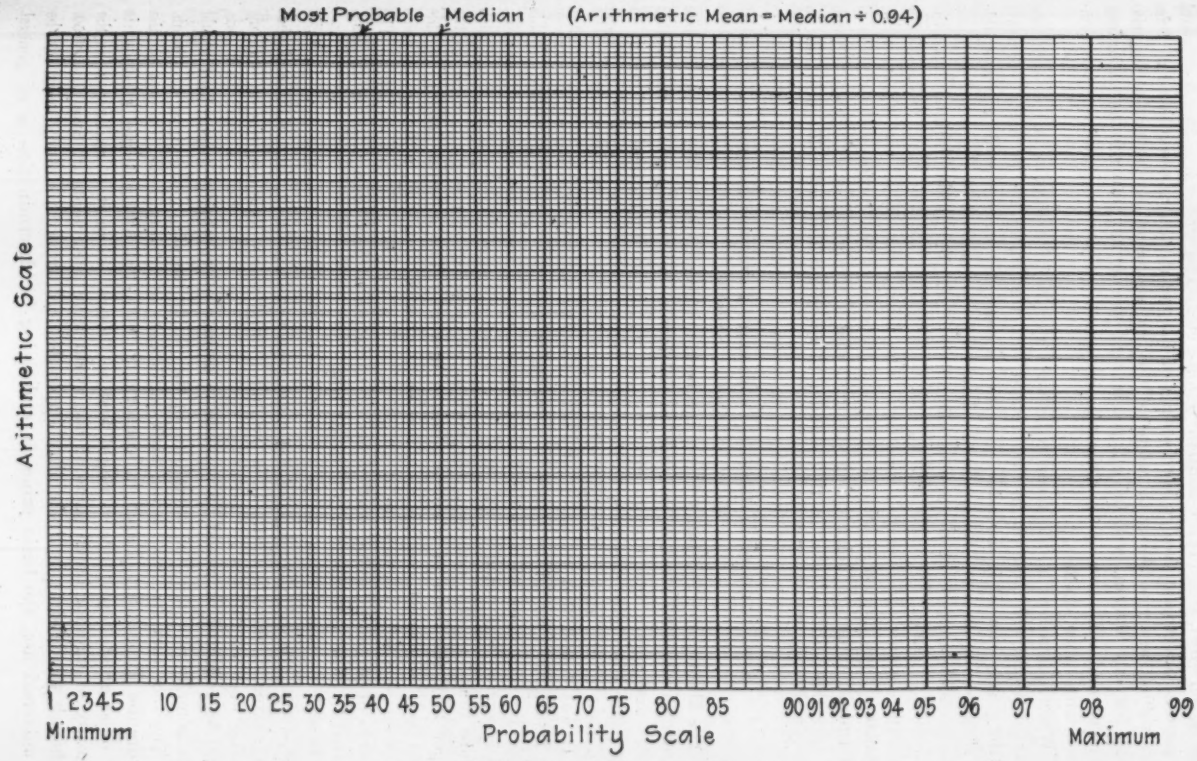

FIG. 4.—HYDRAULIC PROBABILITY PAPER.

that is, acre-feet, second-feet, million gallons, etc. The writer has found it useful to express the stream flow in percentages of the mean, and to use a scale of 1 in. equals 40%, as by so doing the slope of the line representing the probable variations for any stream, will be independent of the actual size of the quantities involved and will show at once the relative extent of such variations when compared with other streams.

When the record of the measured yearly run-off of a stream is less than ten years, a slight modification in the method of determining the probable variation of the yearly run-off is desirable. In such a case, if longer records are available on neighboring streams, these records should first be plotted in the usual manner. The probable wetness or dryness of each individual year is determined from the probability "curve" for each stream, and these values for the same years on the separate streams are averaged together. The yearly run-off of the stream to be studied is then plotted against the probable wetness of the corresponding years as determined from the neighboring streams; and a straight line is drawn through or near the points plotted in this manner to represent the probable variations in the yearly run-off.

Application of Hydraulic Probability Paper to Records of Many Individual Streams.

The Hydraulic Probability Paper has been applied by the writer to the records of many individual streams, not only in California, but in all parts of the United States and also a few foreign streams. It would be impractical to include in this paper the plates necessary to illustrate these applications; however, a general statement of the results obtained will be of value.

It was found that in practically all cases the records of streams plotted on the Hydraulic Probability Paper in accordance with the method described in this paper lie very close to a straight line, regardless of their geographical location. An exception was noted in the case of intermittent and ephemeral streams such as are found in arid regions. The records of such streams when plotted on the Paper took the form of a curve concave upward. The streams falling in this class had, in general, coefficients of variation greater than 0.80, although in some cases streams with coefficients of variation as low as 0.60 fell in this class. The records of streams having coefficients of variation of less than 0.50 in all cases plotted in close agreement to a straight line.

It will be noted from Table 1, that of the thirty-five streams used as basic data, there are only five having variation coefficients greater than 0.60 and that the maximum value is 0.81. It is difficult to find many long records on streams having high coefficients of variation (greater than 0.80), but if sufficient data could be collected, it would be possible to deduce a probability paper that would satisfactorily represent the variations in the yearly run-off of this type of stream.

The fact that streams having high coefficients of variation do not plot on the Hydraulic Probability Paper in conformity with the straight-line relation, is not of as great importance as might first be thought when it is realized that the majority of streams in all parts of the world have coefficients of less than 0.50. Although the Hydraulic Probability Paper deduced from this study may not be of universal use, the basic principle underlying its determination is of general

application; that is, the probable future variations in yearly run-off or any other natural phenomena should be determined only from a study of the same class of phenomena and should not be based on a forced agreement with the normal law of error.

Conclusion.

The writer believes that the asymmetrical distribution of the yearly stream flows is probably of universal application. Further study, however, which would include a larger range of data is necessary before many points are definitely proven. For example, the probability curves, as derived from the data used, might be modified to some extent with further investigations; while the exact location of the most probable value and the relation between the median and the arithmetic mean are points that can be definitely determined only by further study.

This paper, however, has shown the possibility of determining the yearly run-off of a stream without resorting to the much discussed rainfall and run-off relation. The increasing number of stream-flow measurements available each year both in California and in all parts of the United States makes possible such a treatment as has been presented in the preceding pages. It is hoped that in a short period of years, sufficient gaugings of streams in all parts of the country will have been made, so that dependence will not have to be placed in the uncertain relation between rainfall and run-off, and that the consideration of this question will be assigned the secondary position where it rightfully belongs.

The Hydraulic Probability Paper offers a graphical method of obtaining an approximate solution of the probable minimum and maximum yearly stream flow. A knowledge of the minimum flow, even if not exact, is of value in determining the quantity of storage which should be provided. The exact determination of the storage requires special treatment; however, since the variations in the storage requirements are a function of the variations in the run-off, a correct solution of the latter is essential. For irrigation purposes, it is not generally customary in California to store water over from one year to another, so that in this case an indication of the probable maximum shortage of water and the probable percentage of time that some shortage will be experienced should be useful in determining the reliability of a source of water supply for irrigation needs.

Although no thorough investigations have been made, it is believed that the law of probability which has been found to govern the variations in the yearly run-off, can be made to apply, with perhaps some modifications, to the variations in annual precipitation and in maximum yearly flood flows from a drainage area.

The writer wishes to express his appreciation to Edwin Duryea, M. Am. Soc. C. E., for many helpful suggestions during the preparation of this paper. The addition of the coefficient of variation to the record of each stream included in Table 1, was made at the suggestion of Allen Hazen, M. Am. Soc. C. E.

DISCUSSION

ALLEN HAZEN,* M. AM. SOC. C. E. (by letter).—This paper is an important contribution to the study of probability methods applied to flow and storage data; and it is by these methods that these data can be best studied. Rainfall and stream flow may be considered as chance phenomena; meaning by chance, not that the phenomena occur without cause, but that so many causes are at work that the influence of each cannot be determined. Probability methods are applicable when there are many inseparable causes at work.

In any series of numbers representing data of this kind, there is incessant variation. No series of terms is ever repeated. If the run-off for twenty years at a given station is known, it is certain that the run-off for the next twenty years, taken year by year, will not be the same. The mean may approach more or less closely to the old mean, but the individual years will vary, and especially the sequence of high years and low years.

If such a record of any length is available, and it is required to forecast future flows, probability methods furnish the best means of arriving at an approximate solution. If the data followed exactly the normal law of error, the analysis would be easy and certain. The data do seem to follow a law of error, but not the normal law. The divergence, as pointed out by the author, is more or less regular, and, for a given kind of data, can be approximately estimated.

The shortness of the records available is a handicap. Thirteen-year records, or even thirty-year records, are too short to give an accurate idea of the whole situation; even the hundred-year record suggested by Mr. Hall, though better than any now available, is not long enough to serve as a basis for estimating the chances of recurrence of extreme conditions. Bringing records of many streams together may aid in determining such possibilities. The study of more or less similar data, that is, rainfall data, in which series of more terms are available, may aid in understanding the conditions under which extreme values may be expected.

Data that diverge systematically from the normal law of error are sometimes referred to as producing skew curves. Normal curves have equal sides. Skew curves have unequal sides. Applied to stream-flow data, it is found that more than half the terms are below the mean, and the greatest terms exceed the mean by a larger amount than the smallest terms fall short of it. With the normal law of error, the mean is in the middle and the divergencies up and down are equal in both number and magnitude. Mr. Hall's paper may well be called a study of skew curves.

Data represented by skew curves are capable of being handled by mathematical analysis, but the methods are complicated. Textbooks on probabilities are available, which describe the necessary procedure; but for stream-flow data there are two reasons why the mathematical analysis is not advisable: First, the difficulty of applying the methods is too great for some engineers; and, second, and more important, such short-term records as are available are not long enough to serve as a basis for a conclusive determination of the shape of the curve.

The author speaks of his conclusion that variations in the annual run-offs do not follow the normal law of error, and says that this conclusion is as justifiable as

* New York City.

one that data which, when plotted on logarithmic cross-section paper do not lie approximately in a straight line, could not be represented by an exponential equation. Both conclusions are unquestionably sound, but notwithstanding this fact, the greatest use of logarithmic cross-section paper is perhaps to aid in understanding data that do not quite follow the exponential relation. If data do not vary too widely from such a relation, they can be satisfactorily plotted. The line is not straight, but has a moderate curvature, corresponding to the divergence from the exact relation. If any such data follow exactly the exponential relation, logarithmic paper is not necessary, for other rapid means of calculation are available. This applies also to data approaching, but not exactly following, the normal law of error.

As the writer pointed out in 1913,* data of the kind under discussion are represented by skew curves, with the result that lines drawn to represent them on probability paper are concave upward. The writer then used a series of factors (page 1592), deduced from the data which he was then using to aid in plotting these lines, and stated: "It is clear that using the normal law of error in a graphical way, with probability paper, eliminates errors growing out of the unequal variation above and below the mean."

Mr. Hall now proposes to accomplish the same result by distorting the probability paper so as to make it correspond with the degree of skew of his actual data. The distortion arrived at and used may be compared with the factors used by the writer in 1913.* The author's method seems to accomplish the identical purpose in another way; and the method that he proposes may be easier and more advantageous in many cases.

Both these procedures are based on the assumption, which perhaps has not been stated clearly, but which is nevertheless implied, that the amount of skew is so nearly constant that, for the purpose of this procedure, variations in it may be overlooked. There certainly are variations. Whether or not they may be overlooked remains to be discussed. The writer stated in 1913† that other factors differing more or less from the factors used would be found from other similar data; and Mr. Hall suggests that another kind of paper with a different degree of distortion might be prepared for use with data from another type of stream. This raises the question as to how much variation in skew is to be expected in different series of the same kind of data.

In order to discuss this question, one must have a measure of skew that can be applied to each series of data. The writer has taken for the purpose of study the mean of the cubes of the variations, divided by the cube of the standard variation. This value he will call the coefficient of skew. The coefficient of skew may be conveniently calculated at the same time as the coefficient of variation. Two extra columns are required. To make the procedure clear, the calculation for a representative record is given in Table 3.

The coefficient of skew, thus computed, is subject to the objection that there is a tendency for its value to increase with the number of terms in the series; but, even so, it is the most practical measure of skew that the writer has so far found. These coefficients have been calculated by the writer for fifty of the best records of annual flow in his files, and the results are shown in Table 4.

* "Storage to be Provided in Impounding Reservoirs for Municipal Water Supply," *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1590.

TABLE 3.—SAMPLE TABLE SHOWING METHOD OF CALCULATING COEFFICIENT OF VARIATION AND COEFFICIENT OF SKEW.

Kings River near Sanger, Cal.

Year.	Mean daily discharge, in cubic feet per second.	Mean daily discharge, in cubic feet per second, arranged in order of size.	In terms of mean.	Difference from 1, Column (4) — 1.	Square of difference, Column (5).	Cube of Column (5).
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1915	2 510	4 503	1.733	+ 0.733	0.537	0.394
1914	3 522	4 337	1.669	+ 0.669	0.448	0.300
1913	1 301	3 909	1.505	+ 0.505	0.255	0.129
1912	1 337	3 881	1.494	+ 0.494	0.244	0.121
1911	3 909	3 799	1.462	+ 0.462	0.214	0.099
1910	2 459	3 522	1.356	+ 0.356	0.127	0.045
1909	3 881	2 887	1.111	+ 0.111	0.012	0.001
1908	1 423	2 554	0.983	— 0.017	0.000	0.0
1907	3 799	2 510	0.966	— 0.034	0.001	0.0
1906	4 503	2 459	0.946	— 0.054	0.003	0.0
1905	1 975	2 403	0.925	— 0.075	0.006	0.0
1904	2 403	2 334	0.898	— 0.102	0.010	— 0.001
1903	2 334	2 141	0.824	— 0.176	0.031	— 0.005
1902	2 141	1 975	0.760	— 0.240	0.058	— 0.014
1901	4 337	1 782	0.686	— 0.314	0.099	— 0.031
1900	1 782	1 688	0.649	— 0.351	0.123	— 0.043
1899	1 685	1 423	0.548	— 0.452	0.204	— 0.092
1898	1 217	1 337	0.515	— 0.485	0.235	— 0.114
1897	2 887	1 301	0.501	— 0.499	0.249	— 0.124
1896	2 554	1 217	0.468	— 0.532	0.283	— 0.150
Total	2 598*	51 959	19.999	.....	3.139	0.515

$$C\,V = \text{Coefficient of variation} = \sqrt{\frac{\text{Sum of squares}}{N-1}} = \sqrt{\frac{3.139}{19}} = 0.407$$

$$C\,S = \text{Coefficient of skew} = \frac{\text{Sum of cubes}}{(N-1) \times (C\,V)^3} = \frac{0.515}{19 \times 0.407^3} = 0.41$$

* Mean for 20 years.

In Table 4, the first twenty-five items may be classed as Eastern and the remainder as Western streams. The averages of the two groups separated in this manner are shown in Table 5. The averages of Mr. Hall's records are also shown for comparison.

The average skew given for Mr. Hall's data was obtained by calculating the skew for a theoretical twenty-five-year term, scaled from his hydraulic probability paper. It will be seen that the average result from his data corresponds closely with the Western data from the writer's collection.

It is apparent from this comparison that the coefficients of variations are on an average more than twice as high in the West as in the East; and that the coefficients of skew are greater, but not in the same ratio.

In order to determine whether the coefficient of skew increases with the coefficient of variation, the data were re-arranged in the order of size of the coefficients of variations and then averaged without regard to geographical position in groups, with the results shown in Table 6.

TABLE 4.—SUMMARY OF RUN-OFF RECORDS.

Stream.	Place.	Number of years in record.	Last year in record.	Area, in square miles.	Mean run-off, in inches.	Coefficient of variation.	Coefficient of skew.
Kennebec	Waterville, Me.	24	1916	4 270	22.8	0.23	—0.06
Cobbosseecontee	Gardiner, Me	25	1915	220	19.4	0.24	0.12
Androscoggin	Rumford Falls, Me.	22	1916	2 220	22.2	0.16	—0.30
Merrimac	Lawrence, Mass	37	1916	4 560	20.1	0.24	—0.01
Mystic Lake	Boston, Mass.	20	1897	27	19.8	0.27	0.25
Cochituate	South Framingham, Mass.	56	1918	19	19.2	0.26	0.48
Sudbury	" " "	44	1918	75	20.5	0.28	0.43
Wachusett	Clinton, Mass.	22	1918	109	22.2	0.21	0.63
Nashua	Holyoke, Mass.	21	1917	13	26.2	0.21	0.41
Connecticut	" "	20	1899	8 390	19.6	0.21	0.13
Hudson	Mechanicsville, N.Y.	29	1916	4 500	23.5	0.16	0.69
Croton	Croton Dam, N. Y.	45	1912	375	23.0	0.24	0.41
Pequannock	Newark, N. J.	27	1918	62	27.4	0.22	0.84
Neshaminy	Philadelphia, Pa.	28	1912	139	22.6	0.22	0.62
Perkiomen	" "	28	1912	152	22.6	0.22	0.56
Tohickon	Philadelphia, Pa.	29	1912	102	27.7	0.27	0.51
West Branch, Susquehanna	Williamsport, Pa.	21	1916	5 670	22.2	0.16	0.47
Susquehanna	Harrisburg, Pa.	26	1916	24 100	21.1	0.18	0.63
Ohio	Wheeling, W. Va.	21	1905	23 800	22.7	0.19	0.81
Gunpowder	Baltimore, Md.	29	1911	308	19.2	0.30	0.90
Monocacy	Frederick, Md.	20	1916	660	22.0	0.28	0.81
Potomac	Point of Rocks, Md.	20	1916	9 650	14.3	0.25	0.00
Tennessee	Chattanooga, Tenn.	42	1916	21 400	25.1	0.22	0.17
"	Florence, Ala.	22	1916	30 800	22.4	0.23	—0.25
"	Johnsonville, Tenn.	26	1915	38 500	21.6	0.24	—0.29
Winnibigoshish	Minnesota	30	1914	1 442	5.5	0.43	1.21
Sandy Lake	"	19	1914	421	6.8	0.59	0.67
Leech Lake	"	30	1914	1 163	4.7	0.47	0.25
Pine River Reservation	"	28	1914	452	7.8	0.36	0.90
Mississippi	Pekagama	30	1914	3 265	5.5	0.45	0.36
Mississippi	St. Paul, Minn.	25	1916	35 700	4.3	0.42	0.17
Arkansas	Canon, Colo	24	1918	3 060	3.4	0.24	—0.54
Colorado	Austin, Tex.	20	1917	37 000	0.75	0.57	0.97
Rio Grande	Del Norte, Colo.	23	1913	1 400	9.7	0.40	0.52
"	San Marcial, N. Mex.	23	1917	32 000±	0.70	0.57	0.44
Rio Grande	El Paso, Tex.	23	1915	40 000±	0.41	0.75	0.65
Provo	Utah	23	1916	640	8.4	0.42	—0.44
Bear	Collinston, Utah	26	1916	6 000	4.5	0.33	0.35
Spokane	Spokane, Wash.	24	1916	4 000	24.8	0.23	0.34
Willamette	Albany, Ore	24	1915	4 860	40.3	0.26	0.64
Columbia	Dalles, Ore.	37	1915	237 000	13.0	0.21	0.26
Tuolumne	La Grange, Cal.	20	1915	1 500	26.2	0.41	0.51
Kern	Bakersfield, Cal.	22	1915	2 345	5.8	0.56	1.23
Kings	Sawyer, Cal.	20	1915	1 742	20.3	0.41	0.41
Sacramento	Jellys Ferry, Cal.	20	1915	10 400	18.1	0.29	0.14
San Leandro	Oakland, Cal.	36	1914	42	8.1	0.66	0.24
Crystal Springs	San Mateo, Cal.	30	1919	36	11.1	0.69	1.00
Alameda	Sunol, Cal.	30	1919	620	4.8	0.73	1.10
San Gabriel	Azusa, Cal.	22	1917	222	10.6	0.82	0.75
Sweetwater	San Diego, Cal.	31	1918	186	1.6	1.90	8.60

TABLE 5.—AVERAGES.

Record.	Number of years.	Coefficient of variation.	Coefficient of skew.
25 Eastern	28	0.23	0.36
25 Western	25	0.53	0.63
35 Hall's	18.4	0.48	(0.62)

TABLE 6.—AVERAGES.

Term numbers.	Coefficient of variation, CV	Coefficient of skew, CS	$\frac{CS}{CV}$
1 to 20 inclusive	0.21	0.26	1.24
21 to 30 "	0.27	0.46	1.70
31 to 40 "	0.41	0.42	1.03
41 to 45 "	0.59	0.71	1.20
46 to 50 "	0.97	1.42	1.46

It is apparent from Table 6 that on an average the coefficient of skew increases with the coefficient of variation and approximately in proportion to it.

In order to ascertain whether there was any clear-cut relation between size of catchment area and skew, the data were arranged in the order of catchment areas and averaged in groups, with the results shown in Table 7.

TABLE 7.—AVERAGES.

Items.	Drainage area, in square miles.	Coefficient of variation, CV	Coefficient of skew, CS	$\frac{CS}{CV}$
1–10	62	0.33	0.54	1.63
11–20	360	0.58	0.86	1.48
21–30	2 239	0.38	0.45	1.18
31–40	6 230	0.24	0.27	1.12
41–50	52 080	0.36	0.36	1.00

These few data do not show any well marked indications that the coefficient of variation, CV, is either greater or smaller for large streams; but they do indicate that the coefficient of skew, CS, is distinctly less for large streams. The data for the larger streams approach more nearly to the normal law of error.

The author's hydraulic probability paper will show, as a straight line, data that have a coefficient of skew of 0.62. If the coefficient of skew is greater than that, the data will plot concave upward; if it is less, they will plot convex upward. For Western data, bearing in mind that this is an approximate process, one kind of paper may be sufficient, but if the method was to be used for Eastern data, it is clear that a paper with less skew should be provided. The coefficients used as a basis for plotting curves by the writer in 1913* corresponded approximately with a skew coefficient of 0.37 and thus with the degree of skew now determined as an average for Eastern data.

Mr. Hall's method, therefore, is not to be regarded as altogether general. To make it general, it would be necessary to provide a number of kinds of paper for different degrees of skew; and it is suggested that the graduations could be calculated to correspond with theoretical skew curves, data for which could be obtained by calculations from values found in textbooks on the theory of probability. It may be stated that a graduation so calculated for a coefficient of skew of 0.62 would differ but little from that prepared by the author from his data.

For his own use, to aid in drawing smooth curves when points are few and irregular at the ends, the writer is now using a series of factors for curves with different degrees of skew. These factors correspond to those which he used in

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

1913*, but the scheme has been developed and extended to cover varying degrees of skew. The factors used, in tabular form, are given in Table 8.

TABLE 8.—SKEW CURVE FACTORS TO BE MULTIPLIED BY COEFFICIENT OF VARIATION, *CV*, AND ADDED TO OR SUBTRACTED FROM MEAN.

Coefficient of skew.	Terms over mean, in percentages.	99% —	95% —	80% —	50% —	20% +	5% +	1% +	0.1% +
0	50.0	2.32	1.64	0.84	0.00	0.84	1.64	2.32	3.09
0.1	49.2	2.24	1.60	0.85	0.02	0.84	1.68	2.42	3.28
0.2	48.4	2.15	1.57	0.85	0.04	0.83	1.72	2.53	3.48
0.3	47.5	2.05	1.53	0.85	0.06	0.82	1.76	2.64	3.70
0.4	46.7	1.96	1.49	0.86	0.08	0.81	1.79	2.75	3.95
0.5	45.8	1.88	1.45	0.85	0.10	0.80	1.83	2.86	4.20
0.6	45.0	1.80	1.41	0.85	0.12	0.79	1.86	2.97	4.48
0.7	44.2	1.72	1.37	0.85	0.14	0.77	1.89	3.08	4.75
0.8	43.4	1.64	1.32	0.84	0.16	0.75	1.92	3.20	5.06
0.9	42.5	1.57	1.28	0.83	0.18	0.73	1.94	3.32	5.37
1.0	41.7	1.50	1.24	0.82	0.20	0.72	1.97	3.44	5.70
1.1	40.9	1.43	1.20	0.82	0.21	0.70	1.99	3.56	6.10
1.2	40.1	1.36	1.16	0.81	0.23	0.67	2.01	3.69	6.50
1.3	39.2	1.30	1.12	0.80	0.24	0.65	2.03	3.82	6.90
1.4	38.4	1.24	1.08	0.78	0.26	0.63	2 04	3.95	7.35
1.5	37.6	1.19	1.04	0.77	0.27	0.61	2.06	4.08	7.80
2.0	33.6	0.96	0.88	0.71	0.33	0.49	2.10	4.70	....
2.5	29.6	0.74	0.71	0.61	0.36	0.34	2.04	5.50	....
3.0	25.5	0.58	0.57	0.52	0.37	0.18	1.95	6.50	....
3.5	21.1	0.45	0.44	0.43	0.35	0.08	1.75	7.60	....

The factors in Table 8 were reached by an approximate procedure that will not be described. They are used as a means of drawing smooth curves and do not replace, but are used as an aid to, the purely graphical procedure. If some member of the Society who enjoys mathematical problems, would work out factors corresponding exactly with theoretical skew curves, it would be appreciated. Table 8 is in convenient form and covers the range needed for practical work.

The practical application of these factors has been tested in plotting all of the fifty records used in Table 8, and, in addition, many other records representing other kinds of data, and they have proved satisfactory in drawing smooth curves easily and accurately where there is considerable fluctuation. Proceeding in this way, it becomes a matter of secondary importance whether the plotting is made on logarithmic probability paper, on arithmetic probability paper, on the author's hydraulic probability paper; on logarithmic paper as Mr. Fuller† did with his flood data before probability paper was available; or, within limits, on any kind of cross-section paper.

As a general rule, the logarithmic probability paper is best where the coefficient of skew, as defined, is greater than the coefficient of variation. Straight lines on this paper represent the data when the coefficient of skew is twice as great as the coefficient of variation. This is the ratio most commonly found in data relating to flood flows. The coefficient of skew for flood flows is distinctly greater, on an

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

† "Flood Flows," by Weston E. Fuller, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 579.

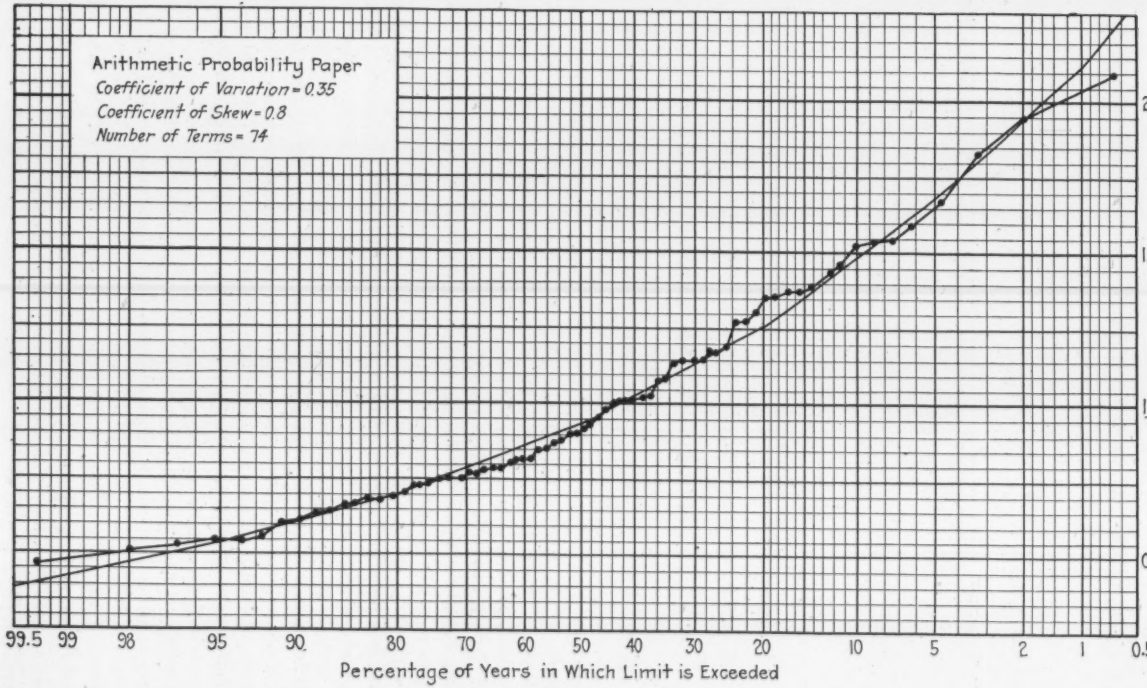
Arithmetic Probability Paper
Coefficient of Variation = 0.35
Coefficient of Skew = 0.8
Number of Terms = 74
2.0
1.5
1.0
0.5
Floods, in Terms of Mean Flood
99.5 99 98 95 90 80 70 60 50 40 30 20 10 5 2 1 0.5
Percentage of Years in Which Limit is Exceeded

FIG. 5.

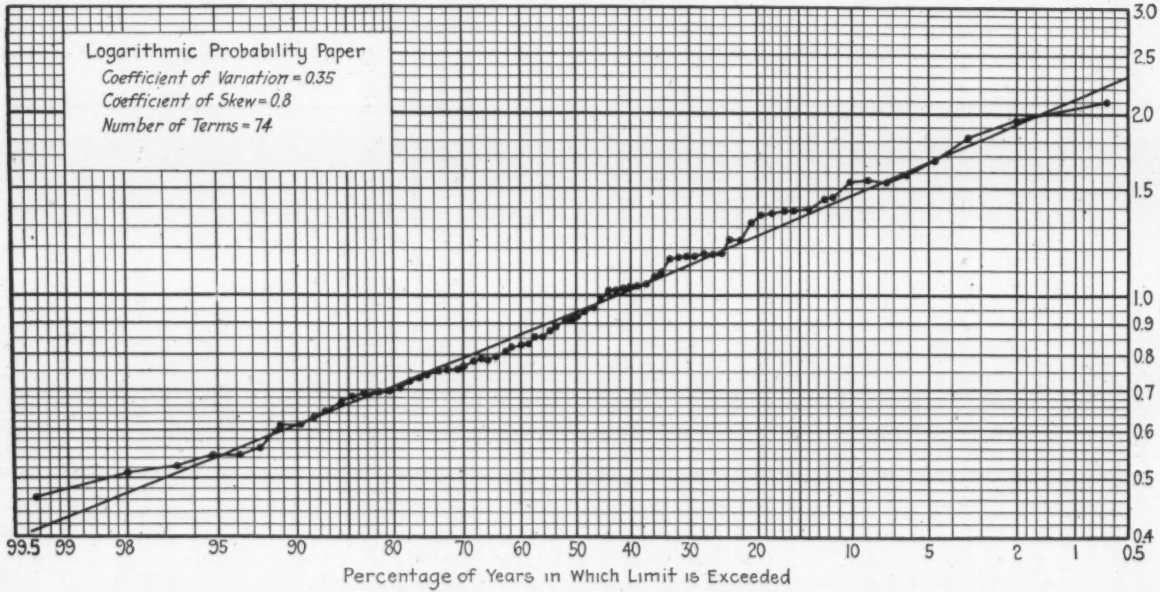
Logarithmic Probability Paper
Coefficient of Variation = 0.35
Coefficient of Skew = 0.8
Number of Terms = 74
3.0
2.5
2.0
1.5
1.0
0.9
0.8
0.7
0.6
0.5
0.4
Floods, in Terms of Mean Flood.
99.5 99 98 95 90 80 70 60 50 40 30 20 10 5 2 1 0.5
Percentage of Years in Which Limit is Exceeded

FIG. 6.

average, than the coefficient of skew for annual flows for the same streams. Rainfall records for the same streams have lower coefficients of variation, but the skews are almost as high as for floods.

The fact that there is so much and varying skew in curves representing these important data adds to the difficulty in making estimates. The writer thinks that part of this variation must be attributed to the shortness of the records, and part of it to errors in the data, and that continued study will gradually lead to a more definite basis for reliable estimate.

In one respect, it is the writer's opinion that the selection of data could be improved. Instead of using so many short records (averaging only 13.4 years), in some of which the probable error is great, according to the author's statement, safer results would be reached by using a few of the longest and most accurate records.

The seventy-four year flood record of the Merrimac River, at Lawrence, Mass., compiled from the records of the Essex Company by R. A. Hale, M. Am. Soc. C. E., and which is shown by Figs. 5 and 6, may be mentioned as an example. As a matter of interest, this record is plotted twice, once on arithmetic probability paper, Fig. 5, and once on logarithmic probability paper, Fig. 6, and in both cases a smooth curve is drawn, based on the previously described factors. For this record, the logarithmic probability paper is clearly the better of the two; but the other plotting is satisfactory to the extent that no great error would be made by using it.

L. F. Harza,* M. Am. Soc. C. E. (by letter).—The author has revived discussion of a subject well worthy of more attention than it received at the time of its introduction by Allen Hazen, M. Am. Soc. C. E., in 1913.†

The chief points brought out by Mr. Hall appear to be an inductive demonstration of the necessity of an unsymmetrical probability paper for the plotting of hydraulic data, in order to cause the points to fall in a straight line, and also the development of such a paper applicable to California streams.

It is quite possible to demonstrate deductively the unsymmetrical character of the form of probability curve which is applicable to the annual run-off data of rivers. For the author's Axiom 2 precludes the application of the accepted probability curve to annual run-off data as follows, "positive and negative errors of the same magnitude are equally probable." This is obviously not the case with run-off data, and especially not with mean annual run-off, which has no fixed upper limit and yet has an absolute lower limit of zero. In the data submitted by the author there are many instances of annual run-offs exceeding 200% of, or 100% above, the mean, whereas, it is obviously impossible for 100% below the mean to be exceeded.

It is also obvious, as a corollary, that the restricted lower range of variation of plotted points will result in a density of points below the mean greater than that above, where more scattering is possible, and, therefore, that the most probable value (or that where the plotted points are most dense) will be below the mean.

The inductive conclusions of the paper are thus so clearly anticipated by deductive reasoning that it would seem to be possible to derive mathematically the equation of a new form of probability curve based on accidental errors of such

* Chicago, Ill.

† "Storage to be Provided in Impounding Reservoirs for Municipal Water Supply," *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

nature as to be unlimited in one direction, and as to approach a definite limit in the other direction in accordance with some definite law, instead of being unlimited in both directions as assumed for derivation of the theory of errors as now accepted. It is to be hoped that this will eventually be done, if possible.

There is at least one class of run-off data, and perhaps there are others, in which the influence of the lower limit of variation does not appear to operate appreciably, if at all, to make the projected line a curve when plotted on standard probability paper. Shortly after the appearance of Mr. Hazen's paper previously referred to, the writer had occasion to apply the method to an investigation of the Columbia

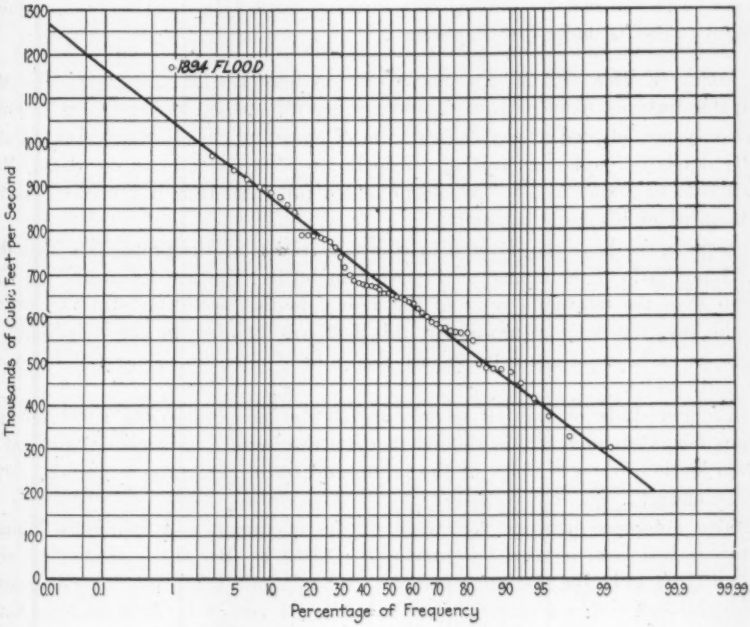

FIG. 7.—PROBABILITY DIAGRAM OF ANNUAL FLOOD PEAKS.

River for power purposes.* The application was to flood peaks and not to annual run-off. Fifty-seven annual flood peaks were available for plotting, and the resulting diagram is reproduced herewith as Fig. 7.

It will be noted that the points in Fig. 7 follow a straight line remarkably well, with no visible tendency toward concavity upward. The straight line shown in the diagram was projected through the points by the method of least squares, thereby eliminating any error of individual judgment. The flood of 1894 was thus shown to be much more improbable than its occurrence once in a 57-year record would indicate.

* "The Columbia River Power Project," L. F. Harza, Technical Publishing Co., San Francisco, Cal.

The writer believes that the influence of the lower limit of variation enters to a less extent to distort the points from a straight line when considering flood flows, because the mean flood flow and individual flood flows are obviously much further removed from the limit zero than the mean annual run-off values. He is not, however, prepared to expect that flood discharges would plot as nearly straight for all streams as they did for the Columbia River. On the other hand, it is quite certain that the Columbia River flood data would plot as a very noticeable curve on the author's unsymmetrical paper.

It is believed that Mr. Hall's paper will prove of much value to the class of data for which it was derived, but that its use will probably require readjustment of the horizontal scale to suit the data in many cases, especially where other than annual run-off is under investigation.

ROBERT E. HORTON,* M. AM. SOC. C. E. (by letter).—This paper contains two valuable features, namely, Table 1, and a suggested method of developing a cross-section paper adapted to any chosen form of equation. The latter feature is not fully developed, and some further consideration of it seems to be desirable. Mr. Hall is to be commended for having presented fully and frankly the fundamental data on which his study is based.

Some space has been devoted to a demonstration of the proposition that the maximum ratio of the annual to the mean run-off has a larger departure from the mean than the minimum ratio of the annual to the mean run-off. The writer would have naturally assumed that this fact was familiar to every hydrologist. If, however, it is not generally known, then a brief discussion along this line may be worth while.

From a study of rainfall records at 153 stations, comprising 6 990 rainfall years, Sir Alexander Binnie showed that the maximum yearly precipitation at a given station averages 153% of the mean, and the minimum year averages 60% of the mean.†

The difference between the range of the maximum and minimum annual rainfalls had been pointed out previously (1883) by Mr. G. J. Symonds,‡ who found 145% to be the maximum and 66% to be the minimum ratio. Similar facts were deduced with reference to both rainfall and run-off in Eastern New York by the late George W. Rafter, M. Am. Soc. C. E., and the writer.§ In this work, it was suggested that the reason for this difference in variability above and below the mean is the obvious impossibility for the annual rainfall or run-off to be less than the mean by an amount over 100%, whereas the departure above the mean is not limited to 100 per cent. In connection with the same study, it was discovered that probability or frequency curves of rainfall and run-off are not symmetrical, and in a letter dated August 11th, 1897, Mansfield Merriman, M. Am. Soc. C. E., suggested to the writer a type of algebraic function, which possibly might be found useful as an empirical equation for skew frequency curves.‖

* Voorheesville, N. Y.

† *Minutes of Proceedings*, Inst. C. E., Vol. CIX (1892).

‡ "British Rainfall," (1883), p. 29.

§ Report of New York State Engr. and Surv., 1896.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 664.

TABLE 9.

Mean rainfall, in inches.	Wettest year, percentage of mean.	Dryest year, percentage of mean.	Range ratio, percentage.
5–30	178	55	123
30–40	154	54	90
40–50	143	64	79
50–60	142	70	72

More recently, Mr. Robert deC. Ward found the relation between the wettest and dryest rainfall years in the United States,* as shown in Table 9, which indicates that the range of variability of annual rainfall increases as the mean annual precipitation decreases.

From a study of some of the long rainfall records in New England, the writer has found the relations between the maximum and minimum yearly rainfall and the mean, as given in Table 10. In this tabulation, the rainfall is shown to be quite uniform, and although the geographical area covered is relatively large and diverse in character, yet the maximum and minimum ratios and range of variability of annual rainfall are surprisingly constant throughout all New England.

Thus, the fact that rainfall above the mean is more variable than rainfall below the mean has been long established. It has generally been assumed *pari passu* that as run-off represents precipitation minus water losses, similar relations hold for stream yield as well as for rainfall, and that conclusion is undoubtedly correct. It may be pointed out, however, that the degree of variability and the law of distribution of run-off may be different from the corresponding factors for rainfall. This incidentally calls attention to the value of the study of the hydrophysical as well as the statistical relations in such matters to avoid serious errors.

TABLE 10.—RANGE OF VARIABILITY OF ANNUAL PRECIPITATION IN NEW ENGLAND.

Station.	Record.			Mean precipitation.	Minimum ratio.	Maximum ratio.	Range ratio.
	From	To	Years.				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Boston, Mass.	1818	1913	96	45.46	0.61	1.49	0.88
Burlington, Vt.	1838	1913	76	32.31	0.71	1.43	0.72
Gardiner, Me.	1837	1913	77	43.30	0.70	1.26	0.56
Cornish, Me.	1857	1913	57	46.19	0.72	1.37	0.65
Cambridge, Mass.	1841	1913	73	43.86	0.69	1.36	0.67
Amherst, Mass.	1836	1913	78	44.15	0.70	1.31	0.61
Providence, R.I.	1832	1913	82	45.01	0.68	1.41	0.73
Waltham, Mass.	1825	1913	89	42.74	0.66	1.45	0.79
Albany, N.Y.	1826	1913	88	37.98	0.69	1.49	0.80
New Bedford, Mass.	1814	1913	100	46.47	0.74	1.41	0.67
Lowell, Mass.	1826	1913	88	41.92	0.67	1.42	0.75

As the writer has shown,† there is throughout England and the Eastern United States, at least, a certain maximum precipitation for which the water losses from a given drainage basin are a maximum. In the case of most streams, the precipitation seldom equals that required for maximum water losses. In the case of other streams, particularly in England, the precipitation sometimes

* *Monthly Weather Review*, September, 1919, p. 631.

† *Engineering News-Record*, March 27th, 1919, pp. 614–616.

exceeds that for which the water losses are a maximum. Under such conditions the annual run-off or yield of a drainage basin suddenly shows a large increase, the rate of increase for high rainfalls being greater than for rainfalls less than that required for maximum water losses.

For the purpose of determining the relation, if any, existing between the drainage area, mean yield, maximum and minimum yield ratios, range ratio, and coefficient of variation for the streams listed in Table 1, the necessary data were tabulated first, in the order of size of drainage areas, and, second, in the order of quantity of mean annual yield. Smoothing the result by averages of groups of five, the values of these various factors shown in Table 11 were obtained. From hydrological considerations it would be expected that the minimum yield ratio would increase with the drainage area and mean yield. Such an increase is indeed generally apparent in Table 11, although not very consistent. All the other factors—maximum yield ratio, range ratio, and coefficient of variability—would be expected to decrease as the drainage area and mean yield increase, and, in general, the group means given in Table 11 show such relations, although there are some inconsistencies, especially with reference to Groups 1 and 2. On the whole, the variation of these three factors with the yield is more consistent than their variation with the drainage area. This would naturally be expected, for the reason that although streams with large drainage basins are well known to be generally less variable than those with small drainage basins, yet it is also true that for a given drainage basin, the annual yield is generally less variable for heavy than for lighter rainfalls. Comparison of the data in terms of mean annual yield takes into account both these factors affecting the range ratio and coefficient of variability.

TABLE 11.—HALL'S DATA OF VARIABILITY OF RUN-OFF OF CALIFORNIA STREAMS ARRANGED IN ORDER OF MAGNITUDE BY MEANS OF GROUPS OF FIVE STREAMS.

Group number.	Mean area, in square miles.	Percentage of Mean.			Coefficient of variation, C_v.
		Minimum annual run-off, Y_l.	Maximum annual run-off, Y_g.	Range ratio, R_r.	
	A—In Order of Area of Drainage Basin.				
1	50.2	46.2	162.4	116.2	0.396
2	215.0	16.1	225.3	209.2	0.656
3	398.4	42.7	188.4	145.7	0.440
4	558.7	34.9	201.4	166.5	0.496
5	772.4	36.5	184.8	148.3	0.508
6	1 466.	36.6	176.8	140.2	0.472
7	48 657.	42.3	181.5	139.2	0.498
	B—In Order of Mean Yield.				
1	112.0	37.1	179.0	141.9	0.446
2	164.8	29.6	222.5	192.9	0.604
3	389.4	32.8	193.7	160.9	0.494
4	689.4	31.1	193.7	162.6	0.526
5	3 085.	43.6	189.2	145.3	0.436
6	1 186.8	41.4	177.2	136.0	0.466
7	46 682.	39.7	165.1	125.4	0.424

It will be noted that the correlation between the coefficient of variation, C_v, and the minimum and maximum yield ratios and range ratio is apparently somewhat

more consistent than the correlation between these various factors and the yield of the drainage areas. Further smoothing the data by combining each pair of groups, and plotting the maximum yield and range ratios in terms of coefficients of variation, the following approximate relations between these factors have been obtained:

$$R_g = \text{Maximum yield ratio} = 350\ C_v^{\frac{2}{3}}$$

$$R_r = \text{Range ratio} = 350\ C_v^{\frac{5}{6}}$$

These empirical equations give a value of the maximum ratio equal to the range ratio for a coefficient of variation equal to unity. Actually, the maximum ratio approaches the value of the range ratio as a limit as the run-off approaches zero. The formulas are rational in form in that they both indicate zero values of the ratios for a coefficient of variation equal to zero.

The average length of the records contained in Table 1 is 13.4 years, so that the values of R_g and R_v given by these formulas may be expected to represent approximately the values of these quantities which will be exceeded on an average once in from 12 to 15 years. The important point is that there seems to be a fairly definite relation between the coefficient of variation, a factor easily calculated from a record of moderate length, and the maximum annual yield and maximum range of variation in yield for a given stream.

In order to determine this relation accurately would require the analysis of run-off records of such great duration as to give approximately correct maximum and minimum values of the annual yield. In view of the fact that the records used are not homogeneous, but vary in length from 9 to 22 years, and, furthermore, as maximum and minimum yield ratios and range ratios are affected not only by the size of the drainage area and amount of precipitation, but also by the physiographic character of the basin, it would be desirable that studies for the determination of these and other relations should be accompanied by statements of the principal hydrological conditions affecting each stream. It is to be hoped that the author will give some data along these lines in his closure, as it seems possible that the inconsistencies in Groups 1, 2, and 3, Table 2, may be due to the fact that some of these smaller drainage areas are either glacier-fed or else more largely fed by melting snow, ground-water, or storage, than the larger streams listed.

As already stated, the writer agrees with the author's proposition that run-off distribution cannot be satisfactorily represented by the ordinary symmetrical or Gaussian law of probability, but requires a skew frequency curve. The writer has already extended this proposition to a large variety of phenomena, including rain intensities for short intervals, used in storm sewer design, maximum rainfall per 24 hours, total rainfall in great storms of 2 to 8 days' duration, maximum flood stages of large rivers, etc. The use of skew frequency curves for rainfall and run-off distribution, however, is by no means new. Such curves for both stream flow and rainfall have been presented by Mr. William Gore* and also by Mr. D. Halton Thomson.†

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXI (1910).

† "Report of Progress in Investigations of Rivers," *Geographical Journal*, December, 1910.

For some time previous to the use of such curves for stream flow and rainfall data, they had been utilized by biologists and actuaries. Recognizing the fact that many classes of biological statistics, such, for example, as the stature of adults, required skew frequency curves for the satisfactory representation of distribution of varying magnitudes, Professor Carl Pearson developed a series of empirical equations for skew and other types of special probability curves.*

Tolley and others have shown that none of Professor Pearson's curves seems to be very satisfactory for representation of rainfall variation.† However, in 1906, Mr. F. Y. Edgeworth developed a generalized law of error.‡ This is an integral function, similar to the Gaussian law of error, but covers both right and left-hand skew curves, the ordinary normal or Gaussian probability curve being a particular case where the coefficient of skew is equal to zero.

The generalized law of error seems to give satisfactory results when applied to positive phenomena dependent on rainfall; however, like the Gaussian law of error, and most frequency curves utilized by biologists and statisticians, it can only be integrated for particular cases. For this reason, it is not wholly satisfactory for engineering work, where integral frequencies, rather than probabilities of individual events, are mostly required, and where, as in studying run-off relations, it is desirable if possible to deduce and compare laws rather than isolated facts. Furthermore, the writer has never felt convinced of the validity of Hagen's or other well-known so-called proofs of the normal law of error.

It is coming to be recognized that the normal or Gaussian law of error is probably nothing better than a close empirical approximation to the true law of distribution of accidental errors which are symmetrical. As noted by the author, it violates one of the three fundamental tenets of the theory of probabilities, in that the normal probability curve is asymptotic to the horizontal axis, whereas actually very large errors do not occur. The writer wishes particularly to emphasize this point in relation to phenomena dependent on rainfall, including the annual yield of streams. The maximum yield of a given drainage basin, the maximum flood discharge from the same drainage basin, or the maximum annual precipitation on the drainage basin would none of them be infinite, even if records covering an indefinitely great length of time were available—yet, according to the Gaussian and generalized law of error, infinite departures from the mean values of the given event should ultimately occur if the record was only continued long enough. The same error exists in certain other empirical forms of frequency curves, such as parabolic expressions deduced by plotting rainfall or run-off values in the order of magnitude on logarithmic or semi-logarithmic paper. As a specific example, Fuller's flood formula§ may be cited.

The writer is prepared to defend the proposition that there are natural conditions which fix definite and finite limits to the magnitude of rainfall, both maximum and minimum, in a given locality, for any chosen period of time, and likewise fix definite limits of magnitude of other phenomena dependent on rain-

* "Statistical Methods," by Davenport (1914).

† "Frequency Curves of Climatic Phenomena," by Howard Ross Tolley, *Monthly Weather Review*, November, 1916, pp. 634–642.

‡ *Journal*, Royal Statistical Soc. of London, Vol. 69 (1906), pp. 497–530.

§ *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564.

fall. Some idea of the natural conditions which limit the amount of rainfall in a given length of time at a given locality can be obtained from a study of meteorological conditions. The specific causes of the limitations in rainfall phenomena are probably complicated and somewhat obscure; however, they are none the less real and definite. If this proposition is true, and the writer has abundant proof that it is, then it follows that any formula used as a frequency or probability equation to represent the distribution of rainfall or other similar phenomena, should intersect the X axis for values of x corresponding to the maximum and minimum possible values of the phenomenon, whereas, ordinarily, frequency curves, as already noted, merely approach the axis of X as an asymptote. Now, it happens that the determination of the absolute maximum and minimum values of phenomena dependent on rainfall are among the most important practical problems of hydrology.

This is a matter to which the writer has given considerable attention. He does not feel that the author's method is entirely satisfactory for the solution of the important problem of determining these maximum and minimum values; furthermore, there is some question whether the varying scale of the author's frequency diagram is correct, since, as already stated, the data on which it is based are not homogeneous, but contain a relatively larger number of intermediate values and a smaller number of extreme values of annual run-off than would be contained in a long single continuous record for one stream. Also, in view of the fact that the annual run-off is subject to various disturbing effects (especially that of extensive storage) which do not affect rainfall, it seems probable that a more consistent relation between average and individual frequency curves would be obtained for rainfall than for run-off.

The writer feels inclined to discourage the assignment of special names, such as "probability paper", "hydraulic probability paper", etc., to systems of cross-section scales which merely reduce a given algebraic function to a rectilinear form when plotted thereon. If the author's contention is correct, as it seems to be, that by the use of a graphical method which he has described, the graph of any function can be reduced to a straight line, then it is evident that an infinitude of such varying scales can be devised; furthermore, it is already well established that there are many different probability curves or functions adapted to different classes of phenomena. The denomination of any particular one of these as "probability paper", therefore, seems pernicious in view of the fact that a large number of others are "probability papers" to an equal degree.

The word rectification is used in the mathematical sense of the determination of the length of an arc of a curve. It seems proper to suggest the use of this term with the additional distinctive meaning of the reduction of the curve to a straight line. By the author's method, the rectification of a curve representing the graph of any given function with specified constants can be accomplished.

One may speak of a function of x and y of a given form, and containing literal constants, as an equation of a given species. If numerical values are assigned to the constants, then the equation becomes an "individual" of that particular species. That which is not obvious, and seems never hitherto to have been presented, is the author's suggestion that having derived the necessary scale for the rectification of an individual curve of a given species, the graphs of all other individual curves

of the same species, whatever the constants may be, will plot on the same system of scales as straight lines. This proposition, if generally true, is of very great importance, and it is to be hoped that the author will present in his closure a general proof of the truth of the method, if available, showing its limitations, if any, and the method of determining the constants in an equation of a given species.

As illustrating what may apparently be accomplished by the use of this valuable method, Fig. 8 has been prepared, showing, first, the graph plotted on uniform scales of the algebraic formula:

$$I = \frac{a}{b + t}$$

and, second, the development of the proper scale and the rectification of this graph by the author's method. In addition, the graphs of several individual curves of the same species have been plotted on the diagram, using the varying vertical scale. These are all straight lines. It will be noted that changing the value of the constant, b, has the effect of shifting a given graph vertically; in other words, all graphs, where a remains constant, are parallel and inclined at a given angle. Likewise, all graphs where b remains constant are parallel, but changing the constant, a, changes the angle of inclination of the lines.

The equation used in plotting Fig. 8 is the function commonly used to represent rain intensities for short intervals, and the semi-hyperbolic scale developed in the diagram by the author's method, makes it possible to plot all such equations, of which there are fifty or more in existence, as straight lines, thus greatly facilitating the derivation of the equation from given data as well as facilitating comparisons of different formulas and the practical use of a given formula.

The writer feels that the author's statement in conclusion to the effect that statistical studies of run-off are to be assigned to a place superior to that of the study of the relation between rainfall and run-off is unfortunate and not justified. Statistical investigations alone may be grossly misleading, if not interpreted in the light of scientific principles. It is only by the study of the causes of phenomena that the principles governing them can be correctly determined. Run-off is merely the residuum of rainfall minus the water losses. One of the first things needed is a more complete knowledge of the principles governing run-off. Having given these, greater reliance can be placed on purely statistical relations. Empirical relations may give excellent results within certain limits, dependent on the nature of the data. The underlying principles may not fix the relations of the quantities sufficiently for practical calculations, but they usually determine the limiting or boundary conditions, and thus serve to prevent the misuse of purely statistical results. There are many cases in point which need not be cited here.

If, as the author states, he has compiled frequency curves of annual run-off for a large number of streams throughout the United States, it would seem desirable that he should present some of these results in his closing discussion, especially as his paper is not accompanied by any single application of his method to an individual stream. However, whether they plot as straight lines by his method or not, the data which he describes, are valuable and could only be duplicated with great labor, and inasmuch as several of the lines could easily be included on a single diagram, it would seem that they should be given for their intrinsic value as well as for illustrating the application of his method.

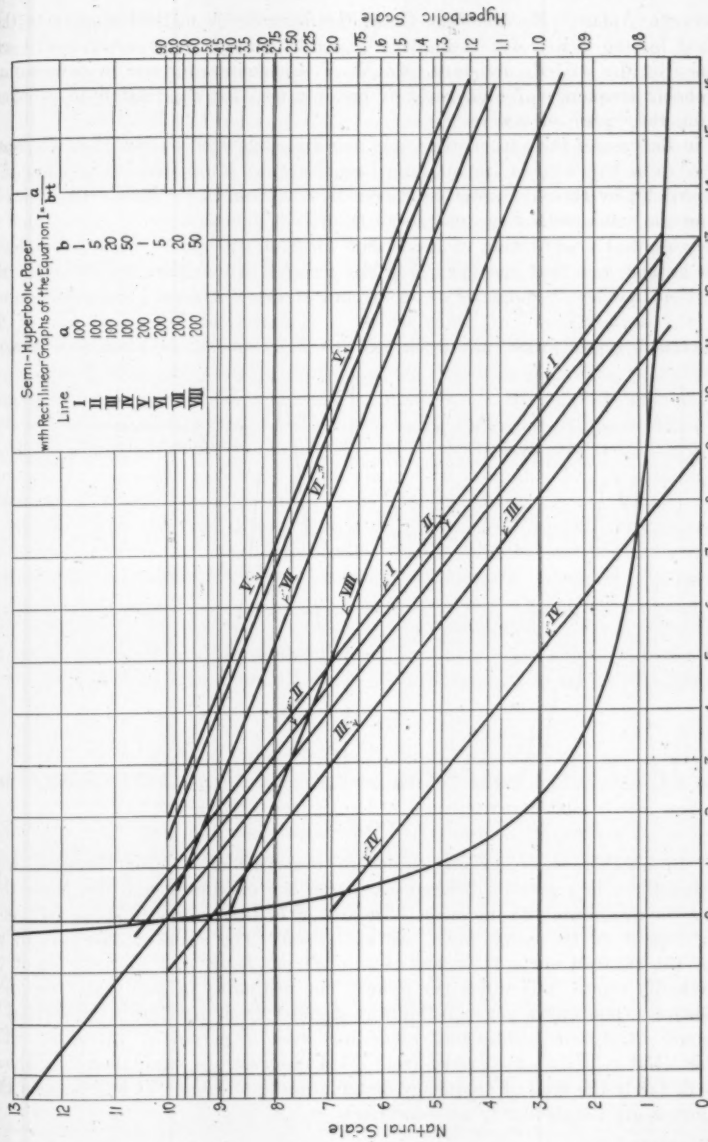
Semi-Hyperbolic Paper
with Rectilinear Graphs of the Equation $I = \frac{a}{b+t}$
Line a b
I 100 1
II 100 5
III 100 20
IV 100 50
V 200 1
VI 200 5
VII 200 20
VIII 200 50
Natural Scale
Hyperbolic Scale

FIG. 8.

KENNETH ALLEN,* M. AM. SOC. C. E. (by letter).—A method similar to that described by the author for estimating probable run-offs from river basins was developed by the writer's assistant, Mr. W. T. Carpenter, for use in determining the probable frequency of high rates of precipitation for application in problems involving the design of sewers.

As in the case of the author, the study was first suggested by Mr. Hazen's paper, "Storage to be Provided in Impounding Reservoirs for Municipal Water Supply",† but an article‡ by Harold G. McGee, Assoc. M. Am. Soc. C. E., supplied the idea of applying the same method of analysis to rainfall phenomena.

The essential modification of the Hazen form of paper both in that developed by the author and that now proposed for rainfall intensities, results from the recognition that the frequencies of occurrence of many natural phenomena do not follow the normal curve of error, which is symmetrical, but some one of the asymmetrical types which are described in many books on statistical theory.

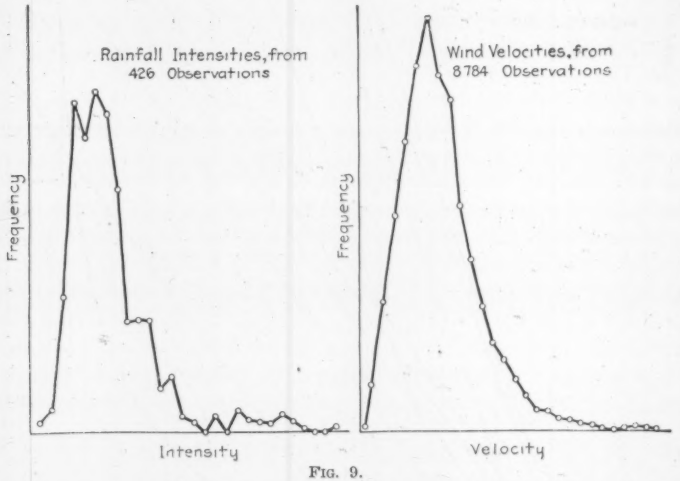

FIG. 9.

Stated in another way: in any series of observations of, for instance, wind velocities, there is a greater difference between the maximum and the mean than between the minimum and the mean. In support of this contention, two frequency curves, Fig. 9, in the usual form, are submitted. The scale of abscissas in the case of the rainfall curve is inches of rainfall per hour, and in the case of the wind-velocity curve it is miles per hour. The ordinates in both cases represent frequencies. The curve for rainfall was derived from maximum intensities in downpours of different duration, extending over a period of 51 years, while that for wind velocity was derived from 8 784 observations, and, therefore, should represent fairly the type of frequency distribution applicable. It will be seen that both curves are unmistakably asymmetrical.

* New York City.
† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.
‡ *Engineering News-Record*, November 13th and 20th, 1919.

Both the author's method and that described by the writer aim at the development of a cross-section sheet on which, if the magnitudes of the desired data (such as intensity of rainfall, in inches per hour) are plotted on a uniform scale of ordinates and the percentage of years in which these rainfalls occur are indicated on the abscissa scale as determined by computation, the resulting curves (in this case for downpours of given duration) will become straight lines, provided they are based on a sufficient number of observations. So that, if such a sheet is prepared from a sufficient number of observations, even if it does not extend over a long series of years, and the existing data are plotted thereon, a straight or slightly curved line can be passed through the points with the assurance that data extending over an infinity of years would approximately conform thereto.

Having once established curves on paper of this kind for downpours of 5, 10, 15, 20, etc., min., intensity curves of the ordinary form for office use are readily plotted with the durations of downpour as abscissas and the intensities as ordinates for any period or percentage of years desired. For example, if the probable maximum for any 5-year period is desired, the procedure is as follows: This maximum will not be exceeded in 80% of the years, hence the series of intersections of the 5, 10, 15, etc., min. duration curves with the 80% vertical give the desired points, through which may be passed a curve showing maximum intensities for different durations that will not be exceeded in 80% of the years.

In this way the dash lines, *A* and *D*, shown on Fig. 10, were constructed. Curve *A* is plotted from the 100th ordinate of the straight-line diagram and, therefore, represents intensities which probably will never be exceeded. It is seen to lie close to Talbot's maximum curve, $I = \frac{360}{(T + 30)}$, in which I = intensity, in inches per hour, and T = duration of downpour, in seconds, and the 45-year curve proposed by the Committee on Rainfall and Run-off of the Municipal Engineers of the City of New York,* namely, $I = \frac{260}{(T + 18)}$.

It will be noted that although the Hazen "Probability Paper" must, in the nature of the probability integral or the normal curve of error—which extends to infinity in both directions—begin with a figure greater than 0 and end with a figure less than 100, and that the author's "Hydraulic Probability Paper" begins at 1 and ends at 99, for reasons discussed by the author, the paper proposed by Mr. Carpenter begins at 0 and ends at 100. This is made possible by the fact that the construction of the paper is empirical rather than theoretical, and has an apparent advantage in that there is a place to plot all the data, whereas in the other papers proposed one has either to omit the figures which are not exceeded in 100% of the years on record or put them in at 99%, or 99.9. If the available data are somewhat limited, the omission of one figure is unfortunate, and there is a measurable error resulting from plotting at 99 which really belongs at 100.

Curve *D*, Fig. 10, is the 10-year curve plotted from the 90% vertical of the straight-line diagram. This is seen to lie close to the curve now in use by the Boroughs of Manhattan and Brooklyn,† $I = \frac{150}{(T + 20)}$, and the curve adopted

* *Proceedings*, Municipal Engrs. of the City of New York, 1913.

† Substituted for the 10-year curve of the Municipal Engineers of the City of New York, $I = \frac{150}{(T+16)}$ by disregarding inlet time.

by the U. S. Housing Corporation during the World War for developments in the vicinity of New York City.

The author has brought out the fallacy of applying the method of least squares to all problems involving probability. This method holds true for all cases where the positive and negative deviations from the mean are equal in magnitude and frequency, as in reading a vernier, but does not hold for phenomena which have a natural limitation or tendency in either direction, as in the case of rainfalls, where the lower limit is zero.

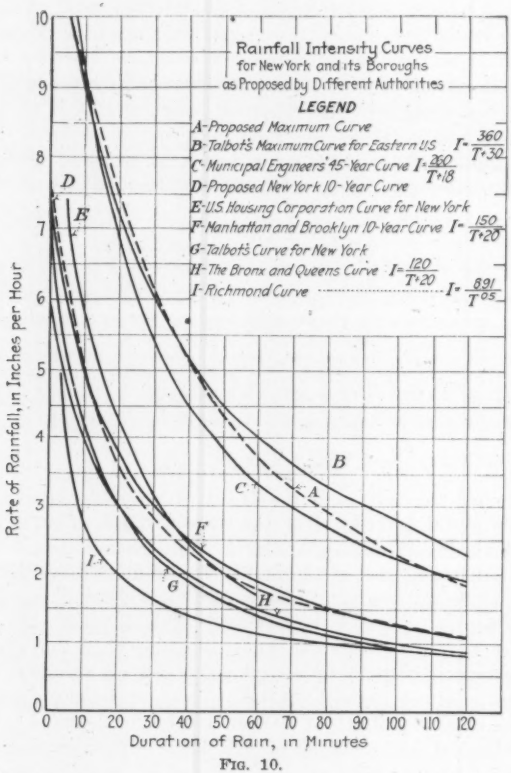

FIG. 10.

C. H. PIERCE,* M. AM. SOC. C. E. (by letter).—The development of some standard method for predicting probable variations in stream flow is certainly to be desired. Hydrology is not an exact science, and probably the factors which govern the flow of a river from year to year are not subject to strict mathematical interpretation. However, if some empirical method can be devised whereby

* Boston, Mass.

future occurrences can be accurately predicted from the results of past observations, a considerable advance will have been made in the science.

It has been shown theoretically and verified by experiment that certain natural phenomena follow definite mathematical laws. With a knowledge of all the factors which may influence a certain event, it should be possible to predict with mathematical accuracy the circumstances of the occurrence of that event. Without a knowledge of those factors, the prediction becomes a matter of chance.

The best method of obtaining advance information in regard to future events is by a study of past occurrences. The probability of an event happening is generally considered to be the ratio of the number of ways (or times) it may happen to the total number of occurrences. If past records have shown that a certain river overflows its banks once in ten years, then the chances are one to ten that there will be an inundation during the next twelve months.

The theory of probability is generally used in the application of the law of error, as expressed in the familiar form:

$$y = k\, e^{-h^2 x^2}$$

It should be noted, however, that in the application of these principles to the analysis of stream flow data, the mathematical interpretation of "error" does not apply to the difference between the value of the measured stream flow for any one year and the value of the mean as determined from measurements over a series of years.

The use of the median instead of the mean when dealing with stream flow records has been advocated by various engineers. The yearly median is a value such that there are an equal number of days with larger values as with smaller; that is, it is the 50% point on the duration curve. One day of low flow offsets one day of high flow; whereas in using the yearly mean, forty or fifty days of low water would be required to balance one day of flood. In the same way, when dealing with yearly values for a period of years, the yearly value for which there are an equal number greater and less would seem to be the one of greatest mathematical probability.

Following this line of reasoning, the writer has prepared tables and diagrams for a number of New England rivers. The general method of procedure as outlined by Mr. Hall has been followed, except that medians have been used instead of means, the percentage departure in each use being that of the yearly median from the median of the period. A total of 120 years' records has been used, and although it is recognized that the use of additional records would be desirable, the limited time available for preparation of the data did not permit of the analysis of other records.

Fig. 11 shows the plotting of points representing the number of yearly medians in each 10% group, when arranged in order of magnitude and expressed as ratios of the median for the entire record on each river, Table 12. A smooth curve of the approximate form of the probability curve has been drawn among the plotted points in such a way as to average the results. The *pseudo* probability curve seems to represent the distribution of the medians in much the same manner as Mr. Hall's curve represents the distribution of the means, although

neither curve actually follows the locus of the points. Similar low places at 110 to 115% appear in both diagrams; also, similar high points at about 130 per cent. An undulating curve like that shown by the dotted line in Fig. 11 would more closely represent the actual occurrences; and it seems probable that an undulating curve of this general shape would come the nearest to a true "hydraulic probability curve."

A trial plotting of medians on the arithmetic probability scales shows that the resulting points come as near a straight line on that paper as the means for the same periods when plotted on Mr. Hall's paper. In either case, some variation occurs, which corresponds to the difference between the conventional curve adopted and the actual plotting of the data.

TABLE 12.—RATIO OF YEARLY MEDIAN TO MEDIAN FOR TOTAL PERIOD COVERED BY RECORDS ON EACH STREAM.

Seasonal year.	Merrimac River, at Lawrence, Mass. Drainage area,* 4 452 sq. miles.	Kennebec River, at Waterville, Me. Drainage area, 4 270 sq. miles.	Connecticut River, at Orford, N. H. Drainage area, 3 100 sq. miles.	Pemigewasset River, at Plymouth, N. H. Drainage area, 615 sq. miles.	Connecticut River, at Sunderland, Mass. Drainage area, 8 000 sq. miles.
1879–80....	1.016	.....	.....	.....	.....
1880–81....	0.775	.....	.....	.....	.....
1881–82....	1.387	.....	.....	.....	.....
1882–83....	0.666	.....	.....	.....	.....
1883–84....	0.808	.....	.....	.....	.....
1884–85....	0.896	.....	.....	.....	.....
1885–86....	1.354	.....	.....	.....	.....
1886–87....	1.529	.....	.....	.....	.....
1887–88....	1.060	.....	.....	.....	.....
1888–89....	2.041	.....	.....	.....	.....
1889–90....	1.965	.....	.....	.....	.....
1890–91....	1.660	.....	.....	.....	.....
1891–92....	1.016	.....	.....	.....	.....
1892–93....	0.885	.....	.....	.....	.....
1893–94....	0.787	0.519	.. ...	.....	.....
1894–95....	0.699	0.568	.....	.....	.....
1895–96....	1.158	0.951	.....	.....	.....
1896–97....	1.376	1.314	.....	.....	.....
1897–98....	1.485	0.970	.....	.....	.....
1898–99....	1.462	0.794	.....	.....	.....
1899–1900..	0.721	1.029	....	.....	.....
1900–01....	0.984	0.940	1.249	.....	.....
1901–02....	1.278	1.215	1.449	.....	.....
1902–03....	1.495	1.039	0.975	.....	.....
1903–04....	0.842	0.813	0.520	0.912	.....
1904–05....	0.754	0.852	1.025	1.039	0.911
1905–06....	1.190	0.686	0.955	1.029	1.304
1906–07....	0.830	1.000	0.905	0.698	0.823
1907–08....	1.572	1.099	1.405	1.261	1.470
1908–09....	0.623	0.656	0.575	0.660	0.519
1909–10....	0.677	1.029	0.775	0.912	0.646
1910–11....	0.546	0.647	0.635	0.747	0.578
1911–12....	0.951	0.813	1.245	1.098	1.176
1912–13....	1.060	1.069	1.385	1.000	1.352
1913–14....	0.951	1.000	0.725	0.884	0.676
1914–15....	0.940	0.872	0.905	1.135	0.833
1915–16....	1.451	1.118	1.325	1.339	1.313
1916–17....	1.113	1.293	1.075	0.873	1.089
1917–18....	0.754	1.010	0.935	0.873	0.833
1918–19....	0.951	1.205	1.505	1.513	1.274
1919–20....	.....	1.089	1.195	1.349	1.099

* Prior to January 1st, 1881, the net drainage area was 4 567 sq. miles; January 1st, 1881, to March 6th, 1898, 4 570 sq. miles; March 7th, 1898, to date, 4 452 sq. miles.

The foregoing discussion of the use of the median instead of the mean is offered as a suggestion to those who are interested in the prediction of stream-flow occurrences. In general, the writer heartily agrees with the line of reasoning advanced by Mr. Hall. Questions in regard to the specific use of the data, or what

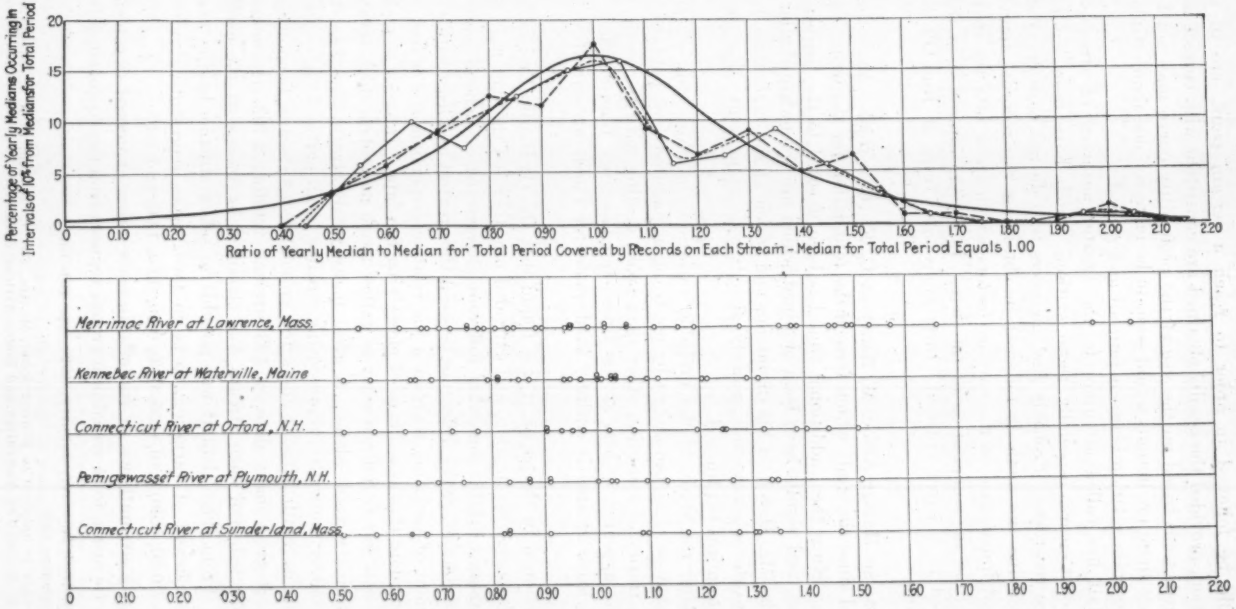

FIG. 11.—PROBABLE VARIATIONS IN YEARLY RUN-OFF.

path should be followed in order to reach a certain point, are of secondary consideration, provided the results obtained are consistent with correct use. Even though predictions based on the use of the probability curve may not be exactly fulfilled, yet those predictions would seem to be more reliable than estimates based on the uncertain relation between rainfall and run-off.

Mr. Hall has called attention to the greater accuracy of stream-flow data during recent years, especially those stations which have been equipped with water-stage recorders. Probably few engineers who do not have occasion personally to examine the methods followed in obtaining the records, appreciate the number and importance of the various factors which enter into the measurement of streams and the compilation of the data. Engineers interested in this subject are referred to a paper entitled "Inspections and Tests of Run-Off Records",* by the writer.

L. STANDISH HALL,† ASSOC. M. AM. SOC. C. E. (by letter).—More than a year has elapsed since the study which formed the foundation for the writer's paper was completed. Since then, additional time has been devoted to the further study of this subject, and results have been obtained which are somewhat similar to those presented by Mr. Hazen in his discussion and which at the same time answer the objection to the original study, namely, that it was not general—that the scope of its application was too limited.

In making the study of the probable variations in yearly run-off, the writer was somewhat handicapped at first by not being acquainted with publications treating of the theory of statistics, other than books dealing with the method of least squares. It appeared to him at the time—and has since been verified—that the method of least squares is entirely inadequate for the treatment of statistics in general.

As mentioned in the paper, it was found that the Hydraulic Probability Paper was not of general application, and extended use disclosed the fact that data from different streams exhibit somewhat different characteristics, depending on the dispersion of the series, or the difference between the greatest and least values observed. The coefficient of variation represents the relative dispersion of a series, so that it is logical to adopt this as a means of classifying the data. In order to make such a classification it was found necessary to gather much more data than were available from California streams alone. A great many yearly stream-flow data, therefore, were collected by the writer, representing streams in all parts of the United States, in Australia, in South Africa,‡ and a few miscellaneous streams from other foreign countries. A great many data on 133 streams, totaling 2 703 stream-years, were thereby collected, and each stream was classified by means of the coefficient of variation. From these data it was possible to select streams having very nearly the same coefficient of variation, so as to form series representing values of the coefficient of 0.20, 0.30, 0.40, 0.50, 0.60, 0.70, 0.80, 1.00, and 1.40.

The yearly stream flows, or run-offs, in each series, expressed as percentages of the arithmetic mean, were sorted into groups representing the frequencies for chosen

* *Cornell Civil Engineer*, January, 1920, pp. 164–169.

† San Francisco, Cal.

‡ The writer is indebted to Elwood Mead, M. Am. Soc. C. E., and to B. A. Etcheverry, Assoc. M. Am. Soc. C. E., for the Australian and South African data.

intervals from the mean, as explained on pages 199 and 200. This grouping formed frequency distributions for each coefficient of variation, which were more or less irregular, due to what is termed the fluctuations of sampling, this irregularity resulting necessarily from the use of a limited amount of data. In order to correlate the data it was necessary to adopt some criterion which would indicate the extent of divergence of the individual series. For this purpose the percentage of terms occurring below the arithmetic mean and the interval in which the mode (or most probable value) occurs, were selected in addition to the coefficient of variation. With these three values properly assigned, the form of the probability curve is more or less fixed.

From an inspection of these series it appeared that the percentage of terms occurring below the mean in each series was some function of the coefficient of variation. Starting 50.5% of the terms below the mean for a coefficient of 0.10, and increasing this percentage by 1.5% for each increase of 0.10 in the coefficient of variation, appeared to give a reasonable solution. Using this rate of increase, the resulting relation is shown in Table 13.

The location of the mode migrates from the interval just below the mean toward the zero end of the curve as the value of the coefficient of variation increases; in fact, for the highest values of the coefficient the mode is located at zero. This will be demonstrated more fully later. Proper values, which would agree with the characteristic just mentioned, were assigned for the location of the mode.

The frequency distributions obtained from the large mass of data collected made apparent the reason for the failure of the Hydraulic Probability Paper to meet general conditions. The frquency distributions assumed three distinct forms, of which the curves shown on Figs. 1 and 2 represent one type.

TABLE 13.—Relation Between Coefficient of Variation and Percentage of Terms Below Arithmetic Mean.

Coefficient of variation.	Percentage of terms occurring below arithmetic mean in actual series.	Assumed percentage of terms occurring below arithmetic mean.
0.10	....	50.5
0.20	48.7	52.0
0.30	55.0	53.5
0.40	55.9	55.0
0.50	55.4	56.5
0.60	59.4	58.0
0.70	58.6	59.5
0.80	61.7	61.0
1.00	68.2	64.0
1.20	....	67.0
1.40	73.5	70.0
1.60	....	73.0
1.80	....	76.0
2.00	....	79.0

On Fig. 12 are shown the three types of frequency distributions encountered in stream-flow data.* Type I represents the form of curve obtained for values of the coefficient of variation ranging from 0.10 to 0.50, the curve having high contact at both ends. After the coefficient of variation reaches the value of 0.60, the

* Compare G. Udny Yule, "Introduction to the Theory of Statistics", 1917 edition, pp. 92 and 99.

lower values of the frequency distribution enter the interval at the zero end of the scale, and the curve leaves the axis at an angle, rising abruptly to the mode. This curve is represented by Type II, and is encountered for values of the coefficient from 0.60 to 1.00. When the mode reaches the interval at the zero end of the scale, the curve assumes the form shown in Type III, which is represented by all values of the coefficient greater than 1.00. From this it is seen that the Hydraulic Probability Paper was based on a consideration of only the curve of Type I, and, therefore, failed to conform with data following the form of distribution shown by Types II and III.

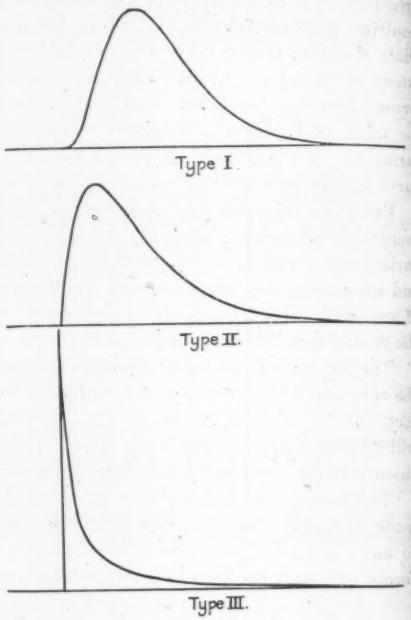

FIG. 12.

By using mathematical methods, it is possible to obtain an equation which will fit any form of frequency distribution. The equations necessary to obtain these curves have already been derived by Professor Karl Pearson and are used extensively in actuarial work for the preparation of life insurance tables.* There are seven types of curves, all of which are obtained from the integration of a function of the general form:

$$\frac{1}{y} \times \frac{d\,y}{d\,x} = \frac{x + a}{b_0 + b_1\,x + b_2\,x^2}$$

The type of curve obtained depends on the roots of the expression, $b_0 + b_1\,x + b_2\,x^2$, and when $b_1 = b_2 = 0$, the integration of the function results in the normal law of error". It is seen, therefore, that the "normal law of error" is merely a special case of the general equation.

The constants necessary to solve the equations indicated were obtained for one or two of the series, but the procedure is extremely laborious, and the results (as referred to by Mr. Horton) are not entirely satisfactory. As the roughness of the original data does not warrant too great precision, a cut-and-try method was substituted, basing the frequency distributions on a 1 000-year series.

By this method the frequency distributions were plotted on cross-section paper, and a frequency curve of the proper type was drawn by eye through the plotted points, placing the mode at the point selected. The desired results were then obtained by a series of adjustments. First, it was necessary to total the frequency distribution in order to see that the number of terms below the mean agreed with

* "Frequency Curves and Correlation", by W. Palin Elderton, 1917 edition.

the values assumed in Table 13, and also that the total for the entire series equalled 1 000. This adjustment was followed by the calculation of the first and second moments of the distribution about the mean. It is necessary that the first moment of the frequency about 100%, should be zero, which is equivalent to stating that the mean of the series should be 100%; and also that the second moment of the frequency about 100% should give the desired value of the coefficient of variation. With the aid of a calculating machine the adjustments could be made very rapidly, only one or two trials being necessary to obtain the desired results.

It is not thought essential to show the computations required in determining each curve; these computations, therefore, will be given (Tables 14 and 15) only for the coefficient of variation of 0.50, and this example may be taken as typical.

The following is a check of calculation and notes, Table 15:

$$\Sigma f_2 \cdot V = -2\,248 + 1\,798 = -450$$

$$\text{Mean } (M) - \text{Arbitrary Value } (A) = -\frac{450}{1\,000} = -0.45^{*} \text{ intervals, or } 4.50\%$$

$$M = 104.5 - 4.5 = 100.0\%$$

$$\sum \frac{f_2 \cdot V^2}{N} = \frac{25\,162}{1\,000} = 25.162$$

$$(C.\,V.)^2 = 25.162 - (0.450)^2\dagger = 24.959$$

$$C.\,V. = 4.996 \text{ intervals, or } 49.96\%$$

TABLE 14.—STREAMS USED FOR PROBABILITY CURVE OF COEFFICIENT OF VARIATION OF 0.50.

Twenty streams are included, totaling 399 stream-years. The $C.\,V.$ for individual streams varies between 0.48 and 0.55; the actual $C.\,V.$ of the series is

$$\sqrt{\frac{99.9442}{398}} = \sqrt{0.2511} = 0.501$$

Name of stream and location of gauging station.	Drainage area, in square miles.	Length of record in years.	C.V.
Goulburn River, at Murchison, Australia	3 966	37	0.48
Coliban River, at Malmsbury, Australia	112	44	0.48
North Fork of Kern River, at Kernville, Cal	1 080	14	0.48
Merced River, at Merced Falls, Cal	1 090	15	0.49
Millers Creek, at Lorella, Ore	270	12	0.50
Truckee River, at Tahoe, Cal	519	18	0.50
Rio Conchos, at La Boquilla, Mexico	.....	10	0.50
American River, at Fairoaks, Cal	1 910	14	0.51
Stanislaus River, at Knights Ferry, Cal	1 051	20	0.51
Mitta River, at Tallangatta, Australia	1 990	38	0.52
King River, at Como, Australia	620	23	0.52
Murray River, at Turrumbarry, Australia	.....	13	0.52
Back Creek, at Nillahcootie, Australia	20	13	0.52
Murray River, at Jingellic, Australia	2 590	28	0.53
Kaweah River, at Three Rivers, Cal	520	15	0.53
Barwon River, at Pollocks Ford, Australia	1 425	12	0.54
San Antonio Creek, near Upland, Cal	.....	15	0.54
Bear River, at Van Trent, Cal	263	14	0.55
Kiewa River, at Kiewa, Australia	434	33	0.55
Little Wimmera River, at L. Lonsdale, Australia	386	16	0.55

* The mid-point of the interval of 100–109, inclusive, or Value A, is 104.5 and not 105; this is because the percentages have been computed only to the nearest whole percentage, thus making the 10% interval really extend from 99.5 to 109.5.

† The standard deviation is a minimum about the mean or 100% in this case. For a complete explanation of the method mentioned see "Introduction to the Theory of Statistics", by G. Udny Yule, Chapters VI, VII, and VIII.

On Fig. 13, the smooth curves represent the computed series, and the agreement of the actual series with these curves is shown by appropriate symbols. The Hydraulic Probability Paper agrees quite closely with the curves for the coefficients of variation of 0.40 and 0.50, these being approximately the average values of the coefficients of the original data in Table 1. It will be noted that in most cases the agreement of the actual series with the computed curves is quite close; the greatest deviations occurring for the curves representing the higher values of the coefficient of variation. This can be explained partly by the lack of sufficient data to determine curves of this type and also by the greater probable errors inherent in distributions of such great dispersion.

The curves shown on Fig. 13 afford a means of estimating the probable variation in the yearly stream flow, or run-off, of rivers by means of the coefficient of variation. These curves were based on the actual measured records of yearly run-off of many streams. Each stream was classified according to its coefficient of variation, and long-term series were formed by combining the records of many streams having approximately similar coefficients. From these actual series, the theoretical curves were deduced by smoothing out the irregularities. Certain curves over very high and very low coefficients of variation have been computed from adjacent series because of the difficulty of obtaining sufficient data for such types of stream.

To apply the diagram to any specific case, it is necessary to determine the coefficient of variation, *C. V.*, of the stream to be investigated, by the equation

$$C.\ V. = \sqrt{\frac{\Sigma\, v^2}{n-1}};$$

where *C. V.* = coefficient of variation, v = percentage of variation of each year from the mean, and n = number of years in record.

When the coefficient of variation and the mean annual run-off have been determined, however, from the measured record of the stream to be studied, or as deduced from the measured records of neighboring streams, the probable variations can be readily obtained from the appropriate curve.

The probable error of the mean of a series which follows the normal law of error is expressed by the formula:

$$e = 0.675\,\frac{C.\ V.}{\sqrt{n}}$$

This formula is of general application and can be used for any type of distribution. The probable error of the coefficient of variation is expressed by the formula:

$$e = 0.675\,\frac{C.\ V.}{\sqrt{2n}}$$

This formula is generally used in all cases, but it is only approximate. The probable error of the mean of a 20-year series having a coefficient of variation of 0.20 is 3.0%, and the probable error of the coefficient of variation is 0.02; while for a series of the same length having a coefficient of 1.00, the probable errors become 15.1% and 0.11. Thus, it is seen that there should be more terms in the series for the larger values of the coefficient of variation in order to maintain the

same relative degree of accuracy, while as a matter of fact the reverse is true due to the difficulty of obtaining data for streams having high values of the coefficient.

TABLE 15.—CALCULATION OF THE THEORETICAL CURVE FOR A COEFFICIENT OF VARIATION OF 0.50.

Interval of percentage of mean. (1)	Frequency of actual series, f.* (2)	Frequency of actual series per 1000 years, f_1† (3)	Assumed frequency per 1000 years, f_2 (4)	Deviation from value A‡, V (5)	Product, $f_2 \cdot V$ (6)	Product, $f_2 \cdot V^2$ (7)	Cumulative percentage of f_2§ (8)
0- 9....	1	3	3	— 10	30	300	0.3
10- 19....	3	7	9	— 9	81	729	1.2
20- 29....	11	28	22	— 8	176	1 408	3.4
30- 39....	19	48	39	— 7	273	1 911	7.3
40- 49....	19	48	61	— 6	366	2 196	13.4
50- 59....	28	70	83	— 5	415	2 075	21.7
60- 69 ...	44	110	95	— 4	380	1 520	31.2
70- 79....	33	83	95	— 3	285	855	40.7
80- 89....	41	103	84	— 2	168	336	49.1
90- 99....	22	55	74	— 1	74	74	56.5
100-109....	29	73	66	0	—2248	.....	63.1
110-119....	26	65	60	+ 1	60	60	69.1
120-129....	26	65	54	+ 2	108	216	74.5
130-139....	15	38	48	+ 3	144	432	79.3
140-149....	18	45	42	+ 4	168	672	83.5
150-159 ...	13	33	36	+ 5	180	900	87.1
160-169....	13	33	31	+ 6	186	1 116	90.2
170-179....	10	25	26	+ 7	182	1 274	92.8
180-189....	8	20	20.5	+ 8	164	1 312	94.85
190-199....	5	13	15.5	+ 9	139.5	1 255.5	96.4
200-209....	6	15	11.5	+ 10	115	1 150	97.55
210-219....	2	5	7.5	+ 11	82.5	907.5	98.3
220-229....	1	3	4.0	+ 12	48	576	98.7
230-239....	1	3	2.0	+ 13	26	338	98.9
240-249....	1	3	1.5	+ 14	21	294	99.06
250-259....	0	0	1.5	+ 15	22.5	337.5	99.2
260-269....	2	5	1.5	+ 16	24	384	99.35
270-279....	0	0	1.5	+ 17	25.5	433.5	99.5
280-289....	1	3	1.0	+ 18	18	324	99.6
290-299....	0	0	1.0	+ 19	19	361	99.7
300-309....	0	0	1.0	+ 20	20	400	99.8
310-319....	0	0	0.5	+ 21	10.5	220.5	99.85
320-329....	1	3	0.5	+ 22	11	242	99.9
330-339....	0	0	0.5	+ 23	11.5	264.5	99.95
340-349....	0	0	0.5	+ 24	12	288	100.00
Total..	399	1 000	1 000.0	.....	+1 798	25 162.0	.....

* Frequency is the number of observations occurring in equi-distant intervals of the variable, in this case, intervals of 10% of the arithmetic mean. The frequencies were determined by actual count.

† Values in Column (3) were obtained by multiplying the frequencies in Column (2) by the ratio $\frac{1000}{399}$.

‡ Value A is any arbitrary value; in this case the interval 100–109 was taken as the arbitrary value about which to compute the moments of the deviations.

§ Values in Column (8) are ordinates of the probability curves for a coefficient of variation of 0.50, shown on Fig. 13.

The curves for the coefficient of variation of 0.10, 1.20, 1.60, 1.80, and 2.00 have been computed, using as a basis the values given in Table 13, and the general form of frequency distribution in the adjacent series. These exterpolated curves should be used with caution and are included to complete the study and to indicate the general form of the probability curves for the extreme ranges of the coefficient of variation.

It would be possible to construct new hydraulic probability papers (as has been mentioned by Mr. Hazen) which would conform to the curve for each value of

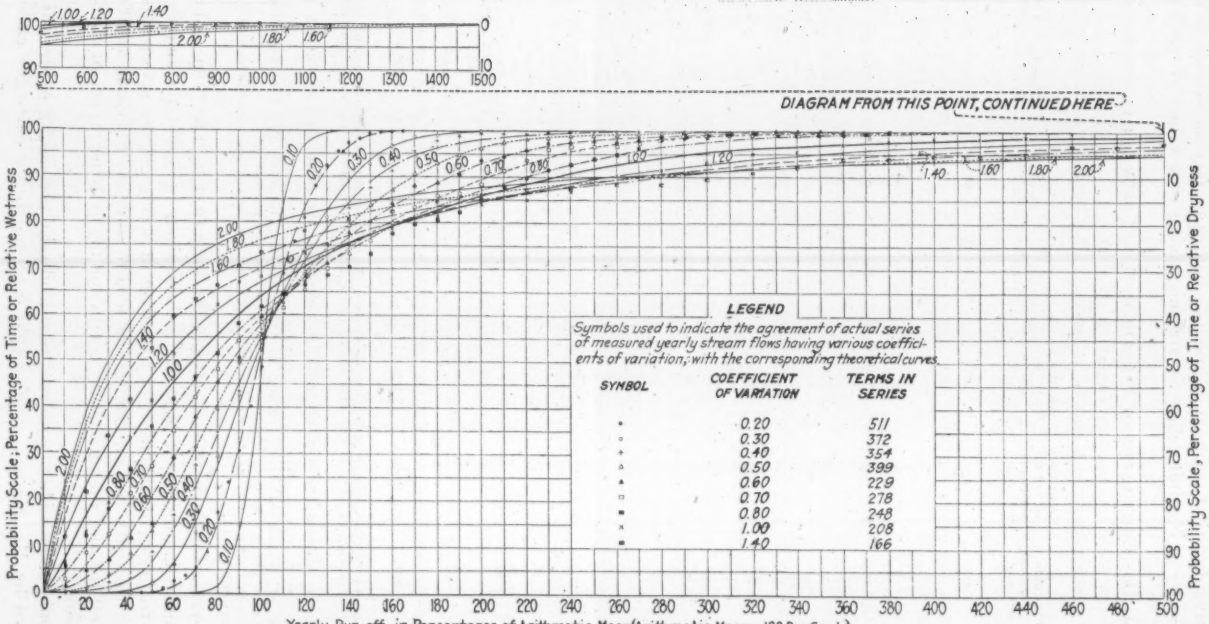
DIAGRAM FROM THIS POINT, CONTINUED HERE
Probability Scale, Percentage of Time or Relative Wetness
Probability Scale, Percentage of Time or Relative Dryness
Yearly Run-off, in Percentages of Arithmetic Mean (Arithmetic Mean = 100 Per Cent.)
LEGEND
Symbols used to indicate the agreement of actual series of measured yearly stream flows having various coefficients of variation, with the corresponding theoretical curves.
SYMBOL | COEFFICIENT OF VARIATION | TERMS IN SERIES
0.20 511
0.30 372
0.40 354
0.50 399
0.60 229
0.70 278
0.80 248
1.00 208
1.40 166

FIG. 13.

the coefficient of variation or, at least, for certain groups of adjacent curves. However, as the curves are now shown, the use of a special probability paper is unnecessary, for if a graphical representation is desired, the data and the curve corresponding to its coefficient may be drawn on ordinary cross-section paper.

Mr. Hazen's discussion is of great value, and introduces some interesting features which merit further study. Of these the principal one is the use of the coefficient of skew as an additional means of classifying the data. There is only one standard accepted by statisticians for a measure of the skewness, which is the one proposed by Professor Karl Pearson:

$$\text{Skewness} = \frac{\text{mean} - \text{mode}}{\text{standard deviation}}$$

Where the mean and the mode are expressed in percentages, the coefficient of variation should be substituted for the standard deviation. The formula used by Mr. Hazen in computing the skewness is known as the approximate formula (with the exception that in the usual formula the factors obtained by Mr. Hazen are multiplied by one-half), and the more exact formula involves the fourth power of the variations.*

It has been found by the writer that the approximate formula with certain types of distribution gives results that vary considerably from those obtained by the more exact formula. However, in dealing with such short records as are available for stream flow, there are certain practical objections to the use of the exact formula, such as the increase in arithmetical work, and the fact that the gain in accuracy is not so great because the higher powers of the variations which must be introduced, become untrustworthy owing to their probable errors being very large. This objection also applies even to the approximate formula when used with short records, and its value (as has been mentioned by Mr. Hazen) is subject to uncertain fluctuations.

In order to illustrate the fluctuations which may occur in the mean, in the coefficient of variation and in the coefficient of skew, progressive values were calculated from two of the longest records available, namely, the 52-year record of Lake Cochituate, at Cochituate, Mass., and the 53-year record of the Murray River, at Mildura, Australia. These results are tabulated in Table 16, and show that although both the mean and the coefficient of variation are subject to fluctuations, in neither case are these fluctuations as great as those exhibited by the coefficient of skew, *C. S.*

It is seen from Table 16 that the value of the coefficient of skew is practically determined by the maximum and minimum variations from the mean, and that the true value of this factor is not obtained until the range of variation is accurately established. In the case of Lake Cochituate, 28 years were required to secure a reliable value; the record of the Murray River does not show such extreme fluctuations in the skewness, but this is due largely to the fact that a year of very great run-off occurred early in the record. If the 45 years from 1871 to 1916 are considered, the mean is 97.2%, the coefficient of variation is 0.46, and the coefficient of skew is 0.48.

* "Frequency Curves and Correlation", W. Palin Elderton, p. 41.

TABLE 16.—PROGRESSIVE VALUES OF THE MEAN, THE COEFFICIENT OF VARIATION, AND THE COEFFICIENT OF SKEW COMPUTED FROM TWO LONG RECORDS.

	LAKE COCHITUATE AT COCHITUATE, MASS.; RUN-OFF FOR SEASONAL YEARS ENDING SEPTEMBER.				MURRAY RIVER AT MILDURA, AUSTRALIA; RUN-OFF FOR CALENDAR YEARS.			
Year.	Percentage of yearly run-off.	Percentage of progressive mean.	Progressive *C.V.*	Progressive *C.S.*	Percentage of yearly run-off.	Percentage of progressive mean.	Progressive *C.V.*	Progressive *C.S.*
1864	107	107	....	....				
1865	112	109.5	....	....	60	60	....	....
1866	80	99.7	....	....	62	61	....	....
1867	114	103.3	....	....	133	85	...	....
1868	126	107.8	....	....	52	76.8	....	....
1869	100	106.5	....	....	55	72.4	....	....
1870	156	113.6	....	....	301	110.5	....	....
1871	74	108.6	....	....	166	118.4	....	....
1872	78	105.2	....	....	....	....	....	....
1873	133	108.0	0.24	0.27	136	120.6	....	....
1874	125	109.6	0.23	0.11	135	122.2	....	....
1875	77	106.8	0.24	0.24	152	125.2	0.61	1.11
1876	105	106.7	0.23	0.26	86	121.6	0.60	1.26
1877	107	106.7	0.22	0.27	53	115.9	0 63	1.35
1878	132	108.4	0.22	0.08	112	115.6	0.60	1.43
1879	116	108.4	0.22	0.11	98	111.0	0.59	1.50
1880	56	105.8	0.25	−0.14	123	114.6	0.56	1.50
1881	80	104.3	0.25	−0.01	63	111.4	0.57	1.57
1882	79	103.0	0.25	0.12	74	109.2	0.57	1.73
1883	54	100.6	0.27	0.03	86	107.9	0.56	1.77
1884	95	100.3	0.27	0.04	53	105.0	0.57	1.87
1885	72	99.0	0.27	0.15	59	102.8	0.58	1.84
1886	118	99.8	0.26	0.08	63	100.8	0.58	1.93
1887	130	101.1	0.26	−0.01	148	103.0	0.57	1.84
1888	108	101.4	0.26	−0.04	93	102.5	0.56	1.88
1889	167	103.9	0.27	0.19	170	105.3	0.54	1.72
1890	133	105.0	0.27	0.11	136	106.6	0.53	1.68
1891	189	108.9	0.30	0.45	141	107.9	0.52	1.55
1892	80	107.0	0.30	0.51	95	107.4	0.51	1.60
1893	85	106.3	0.30	0.56	135	108.4	0.50	1.57
1894	71	105.1	0.30	0.60	198	111.5	0.50	1.34
1895	78	104.3	0.30	0.65	92	110.8	0.49	1.39
1896	125	104.9	0.30	0.60	54	109.0	0.50	1.43
1897	87	104.4	0.30	0.64	52	107.2	0.51	1.43
1898	109	104.5	0.29	0.64	54	105.6	0.52	1.45
1899	120	105.1	0.29	0.59	56	104.1	0.52	1.46
1900	91	104.6	0.29	0.63	94	103.9	0.52	1.53
1901	123	105.1	0 28	0.59	56	102.5	0.52	1.52
1902	121	105.5	0.28	0.56	15	100.2	0.55	1.42
1903	116	105.7	0.28	0.54	57	99.0	0.55	1.43
1904	100	105.6	0.27	0.55	72	98.3	0.55	1.51
1905	76	104.9	0.27	0.60	76	97.7	0.54	1.58
1906	86	104.4	0.28	0.59	160	99.3	0.54	1.41
1907	72	103.7	0.28	0.66	68	98.6	0.54	1.49
1908	99	103.6	0.27	0.67	53	97.5	0.54	1.50
1909	68	102.8	0.28	0.69	122	97.9	0.54	1.53
1910	69	102.1	0.28	0.69	71	97.5	0.53	1.52
1911	47	101.0	0.29	0.60	101	97.6	0.53	1.57
1912	98	100.9	0.29	0.62	50	96.6	0.53	1.57
1913	80	100.5	0.29	0.65	68	95.8	0.53	1.59
1914	101	100.5	0.29	0.65	17	94.3	0.55	1.48
1915	75	100.0	0.29	0.69	81	94.0	0.54	1.55
1916	....	....	....	....	134	95.0	0.54	1.52
1917	....	....	....	....	281	98.5	0.58	1.53
1918	....	....	....	....	183	100.0	0.57	1.46

The writer does not wish to decry the use of the coefficient of skew, as there is undoubtedly a tendency for stream-flow data to show varying degrees of skewness, but the accuracy of the results computed from 30-year or even from 50-year records is open to question, and it is considered that the use of this factor introduces a refinement not warranted by the data available.

In order to investigate this subject further, Table 17 was prepared giving values of the coefficients and also the percentage of terms below the mean and the location of the mode for forty of the longest records available. It is difficult to determine the mode exactly, but it can be found approximately by averaging the terms in the interval or intervals showing the greatest frequency. Some of the streams in Table 17 are the same as those given in Table 4 by Mr. Hazen, but in most of these cases the length of record is different and the re-tabulation will illustrate the fluctuations of the factors being discussed.

The streams in Table 17 are arranged in the order of magnitude of the coefficient of variation, so they were divided into groups of five and all factors averaged, with the results shown in Table 18. It will be noted in general that as the coefficient of variation increases, the coefficient of skew and the percentage of terms below the mean also increase, while at the same time the mode approaches zero. Since the coefficient of skew increases with the coefficient of variation, this feature has been taken care of in the revised probability curves. The percentage of terms below the mean and the location of the mode as determined from these selected data are quite similar to those used in calculating the curves shown on Fig. 13.

The streams were re-arranged in the order of magnitude of the coefficient of skew and the factors again averaged in groups of five, with the results shown in Table 19. As might be expected, the coefficient of variation and the percentage of terms below the mean increase with the skew, but while the mode shows a tendency to decrease, the results are not as satisfactory as in the comparison made in Table 18. In preparing the factors in Table 8, Mr. Hazen has assumed that the percentage of terms above (or below) the mean varies directly with the coefficient of skew. The relation shown in Table 19, although somewhat irregular, agrees fairly well on the average with that adopted by Mr. Hazen.

Referring again to Table 8, it is seen that for any value of the coefficient of variation there is a minimum value for the coefficient of skew, than which a lower value will give negative quantities for the run-off to be expected in extremely dry years; in other words, the coefficient of variation multiplied by the factors in the 99% column must be less than 100 per cent. From this it is seen that in general the coefficient of skew must increase with the coefficient of variation.

An attempt was made to determine whether any relation existed between the mean minus the mode and the product of *C.V.* and *C.S.*, but this was unsuccessful as the values were too scattered to draw any definite conclusion. The streams in Table 17 also were arranged in order of magnitude of the drainage areas and a table similar to Table 7 was prepared. The writer found with his data no uniform decrease in the ratio $\frac{C.S.}{C.V.}$ with the increase in the size of the drainage area as found by Mr. Hazen. The ratio fluctuated in an irregular manner and showed no definite trend.

TABLE 17.—SUMMARY OF RUN-OFF RECORDS.

Stream.	Place.	Number of years in record.	Last year of record.	Area, in square miles.	Mean run-off, in inches.	Coefficient of Variation, *C. V.*	Coefficient of Skew, *C. S.*	Percentage of terms below mean.	Mode as percentage of mean.
Merrimac	Lowell, Mass.	61	1915	4 097	26.5	0.14	0.16	48	100
Lake Champlain	Chambly, Que., Canada	41	1916	8 440	20.4	0.16	— 0.02	51	100
Columbia	Dalles, Ore.	37	1915	237 000	12.1	0.17	0.06	41	101
Redmires	Sheffield, England	29	1907	4	26.2	0.18	0.19	52	101
Tennessee	Chattanooga, Tenn.	39	1913	21 400	25.1	0.23	0.22	49	90
Merrimac	Lawrence, Mass.	36	1915	4 560	20.1	0.23	0.00	54	94
Croton	Croton Dam, N. Y.	32	1899	360	22.6	0.24	0.25	47	100
Lake Cochituate	Cochituate, Mass.	52	1915	18	19.2	0.29	0.69	50	97
Sudbury	Framingham, Mass.	40	1915	77	20.8	0.30	0.07	52	102
Chagres	Gatun, Panama	25	1914	1 320	74.2	0.30	0.86	64	79
Yarra	Warrandyte, Australia	27	1918	972	14.2	0.31	0.24	44	94
Sacramento	Red Bluff, Cal.	25	1920	10 400	16.9	0.33	0.15	48	108
Mitchell	Bairnsdale, Australia	29	1918	1 770	7.3	0.39	0.18	52	104
Tuolumne	Lagrange, Cal.	25	1920	1 500	25.3	0.40	0.46	52	96
Kings	Sanger, Cal.	25	1920	1 740	21.2	0.40	0.53	64	84
Mississippi	St. Paul, Minn.	25	1916	35 700	4.1	0.43	0.42	56	98
Mackenzie	Wartook, Australia	32	1918	29	13.9	0.44	— 0.22	47	121
Goulburn	Murchison, Australia	37	1918	3 966	11.0	0.48	1.14	60	74
Coliban	Malmsbury, Australia	44	1918	112	8.9	0.48	0.41	57	93
Mitta	Tallangatta, Australia	33	1918	1 990	11.0	0.52	1.46	58	74
Murray	Jingellic, Australia	28	1918	2 520	14.0	0.53	1.59	61	74
Rio Grande	San Marcial, N. Mex.	25	1919	32 000	0.67	0.54	0.34	48	90
Kiewa	Kiewa, Australia	33	1918	434	23.2	0.55	2.18	64	79
Murray	Mildura, Australia	53	1918	92 000	1.8	0.57	1.46	62	58
Ovens	Wangaratta, Australia	32	1918	2 090	10.6	0.62	1.90	63	75
Verde	Granite Reef Dam, Ariz.	31	1919	6 000	1.9	0.64	1.26	58	79
Kern	Bakersfield, Cal.	26	1919	2 345	6.4	0.65	1.70	62	69
Murray	Morgan, Australia	29	1919	150 000	1.1	0.65	1.37	62	65
Loddon	Laanecoorie, Australia	28	1918	1 590	2.6	0.68	0.90	58	55
Crystal Springs	San Matteo, Cal.	30	1919	36	11.1	0.69	1.04	57	65
San Leandro	Oakland, Cal.	34	1920	42	9.0	0.72	0.72	53	89
Campaspe	Rochester, Australia	33	1918	1 356	2.7	0.73	0.65	61	35
Alameda	Sunol, Cal.	27	1918	620	4.7	0.74	1.10	63	59
Glenelg	Balmoral, Australia	30	1918	606	3.8	0.76	1.32	63	53
Broken	Goorambat, Australia	32	1918	730	6.0	0.78	1.60	69	45
Salt	Granite Reef Dam, Ariz.	31	1919	6 260	2.5	0.82	1.54	59	35
Avoca	Coonooer Weir, Australia	29	1918	1 029	1.1	0.84	0.52	55	15
Wimmera	Horsham, Australia	30	1918	1 530	1.8	0.86	0.60	57	20
Wimmera	Glenorchy, Australia	30	1918	768	1.4	0.92	0.82	70	26
Sweetwater	San Diego, Cal.	32	1919	181	1.7	1.87	3.66	72	4

TABLE 18.—AVERAGES IN ORDER OF MAGNITUDE OF THE COEFFICIENT OF VARIATION.

Group.	*C.V.*	*C.S.*	Terms below mean.	Mode.
1	0.18	0.12	48	98
2	0.27	0.37	53	94
3	0.37	0.31	52	99
4	0.47	0.64	56	92
5	0.56	1.49	60	75
6	0.66	1.25	59	67
7	0.75	1.08	62	46
8	1.06	1.43	63	20

All this is only illustrative of some of the difficulties encountered in estimating the probable variations of statistics of this nature. The whole procedure is at best approximate, but when this is kept in mind, results can be obtained which will prove of great value and will lead to a better understanding of the nature of the fluctuations of yearly run-off.

TABLE 19.—AVERAGES IN ORDER OF MAGNITUDE OF THE COEFFICIENT OF SKEW.

Group.	*C.S.*	*C.V.*	Terms below mean.	Mode.
1	−0.02	0.26	49	104
2	0.18	0.25	50	101
3	0.33	0.40	50	95
4	0.55	0.65	58	50
5	0.80	0.58	59	59
6	1.17	0.66	60	66
7	1.48	0.62	60	61
8	2.21	0.89	66	54

The writer wishes to call attention to one point with regard to Mr. Hazen's storage paper* which was not brought out in the discussions at that time. It is believed by the writer that the treatment of the combination of the monthly and annual storages used by Mr. Hazen gives undue weight to the depletions of a reservoir during periods of extreme drought. The writer has had occasion to prepare a diagram for the purpose of determining storages required for the development of certain streams in the vicinity of San Francisco Bay, and will describe briefly the method of obtaining the results, both to illustrate the point mentioned and also to give an example of an important application of the Hydraulic Probability Curves. The streams in question have an average coefficient of variation of 0.70, so that the results will apply to any other streams having approximately the same coefficient of variation, *C.V.* In preparing this diagram, certain modifications to Mr. Hazen's methods have been introduced, which will be explained.

Storage may be divided into two parts, which can be treated separately: (1) monthly storage, or that required to equalize the natural flow of the stream during the months of the year; and (2) annual storage, or that required to carry water over from wet years to make it available in dry ones.

Where the annual draft to be used from the stream is less than the flow in the 95% dry years, a diagram similar to Fig. 26 of Mr. Hazen's paper,† is applicable. For streams in the vicinity of San Francisco, having a coefficient of variation of approximately 0.70, the flow in the 95% year would be 18% of the mean (Fig. 13). It is evident that on streams of this type, except for very small developments, annual storage is essential.

In his storage paper mentioned, Mr. Hazen has computed the annual storage required in the 95% dry year (to use a finite example) and has added to this the monthly storage required in the 95% dry year for the same rate of draft. In more recent results‡ Mr. Hazen has used a different method of procedure; but as neither of these methods will apply with the treatment of annual storages used by the writer, he will state his analysis of the problem.

If the yearly run-offs are arranged in order of months of low flow followed by months of flood flow, it is seen that, starting with reservoir full at the beginning of any period, if the total run-off for the year is less than the rate of draft, there will be a depletion of the reservoir, and that this depletion will increase as long as there are successive years of run-off below the rate of draft. Considering a year in

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.
† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1585.
‡ "American Civil Engineers' Handbook", 1920 edition, p. 1197.

which the run-off is less than the draft, it should be remembered that the monthly storage is the excess storage above that needed if the stream flowed at a uniform rate.

If S_m is the maximum drawdown of the reservoir at the close of the dry summer months and S_a is the drawdown at the end of the year, the excess required for monthly storage would be $S_m - S_a$. However, in all cases in order to conclude the drought and start the refilling of the reservoir, a year of run-off above the rate of draft must occur. As any such year might conclude a period of drought, a logical method of allowing for monthly storage would be to add to the annual storages the average monthly storage required during years having flows above the yearly draft. For low rates of draft, the variations in the monthly storage, as illustrated in Fig. 26 of Mr. Hazen's storage paper already referred to, may exceed the results obtained by the previously described method; in such a case the larger storage should be used.

The mean monthly storage for years with yearly flow above the assumed rate of draft has been computed from the measured records of Alameda Creek, Coyote River, Putah Creek, and Cache Creek, and the results are expressed in terms of the mean annual flow. It was found that the monthly storage required for the Coyote River and Putah Creek were somewhat greater than for Alameda Creek and Cache Creek. The results from the former streams were practically identical, and were accepted as representing normal monthly storage. The flow of Alameda Creek (as measured at Sunol Dam) is regulated to some extent by the large underground storage available in the Livermore Valley, while the flow of Cache Creek is undoubtedly regulated by Clear Lake. The storage required for the Coyote and for Putah Creek, which was accepted as the proper allowance for monthly storage to be added to the annual storages, is given in Table 20.

TABLE 20.—NORMAL MONTHLY STORAGE FOR YEARS WITH FLOW ABOVE RATE OF DRAFT.

Yearly draft as percentage of mean flow	10	20	30	40	50	60	70	80	90	100
Monthly storage as percentage of mean flow	2	8	14	21	27	33	39	45	51	57

The annual storages remain to be considered, and these will be treated by a method differing from that used by Mr. Hazen.

If the probability scale shown on Fig. 13 is divided into 100 equal parts and the yearly run-off opposite the middle of each of these intervals recorded, the result will be an ideal 100-year record in percentages of the mean annual flow. This would mean reading the value of the yearly run-off opposite the value 0.5, 1.5, 2.5, * * * 97.5, 98.5, 99.5. However, to determine the annual storage, the order or sequence in which the individual years will occur must be known.

This was obtained by applying the laws of chance, or by using an artificial (drawn) record of 1 000 years as follows:

To apply the laws of chance, the following tacit assumptions must be made:

1.—That the true mean stream flow is a constant; that is, that there is no progressive change in the mean run-off, due to climatic changes. Such changes are

known to extend over geologic ages, but it is not believed that their effect would be noticed in 100 years or even in 1 000 years.

2.—That the variations from the mean are identical for all streams having the same coefficient of variation. There seems to be apparent exceptions to this, but it can only be proven or disproven by long-time records on individual streams; as this was the assumption made in deriving the curves shown on Fig. 13, it will be adhered to.

3.—That the stream flow of each year is independent of the years preceding or following. This assumption appears to be justified from actual records, and was checked by Mr. Hazen* by making drawn records and comparing them with the actual records.

To obtain the artificial or drawn record, the yearly stream flows for the ideal 100-year record were written on separate cards and placed in a box. Cards were drawn from the box, one at a time, and the values recorded. After each card had been drawn, it was returned to the box before the next card was drawn, in order that there might be no correlation between one drawing and the next. The reason for this requirement is apparent, but it is explained more fully in textbooks on statistics.†

A record of yearly stream flows for a period of 1 000 years was made in the manner previously described, and storages were computed for seven rates of draft, assuming the reservoir to be full at the beginning of the period. The drafts selected were chosen by Mr. Hazen's reduction factor,‡ $1 - k$ ($C.\ V.$), using values of k of 0.1, 0.2, 0.4, 0.6, 0.8, 1.0, and 1.2; the corresponding drafts being 93, 86, 72, 58, 44, 30, and 16% of the mean annual flow.

The difference between the methods of Mr. Hazen and the writer must now be carefully noted. Instead of tabulating the storage at the end of each year, as was done by Mr. Hazen, the writer recorded only the maximum storage required during each period of depletion of the reservoir. When the reservoir is drawn for a period of years, the annual storage required at the end of each year depends not only on the stream flow of that year, but also on the depletion of the reservoir at the beginning of the year. Thus, it is seen that the only "events" in the storage series that are independent of each other, are the maximum depletions during each period of drawdown and refilling of the reservoir.

These maximum depletions were then arranged in order of magnitude for the first 100 terms of the series, corresponding to the depletion of the reservoir equalled or exceeded once in 10 years. A slight modification to this procedure was necessary for high rates of draft, where the reservoir may be drawn down for a long period of years. For example, if a reservoir is drawn down for a period of 40 years during a certain period of drought, in figuring the storage equalled or exceeded once in 20 years, such storage will be the maximum for two 20-year periods, or if a period of 10 years is considered, it will be the maximum for four 10-year periods. Such large storages should be given the proper weight.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), pp. 1611–1612.
† "Introduction to the Theory of Statistics", by G. Udny Yule, pp. 335–336.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1594.

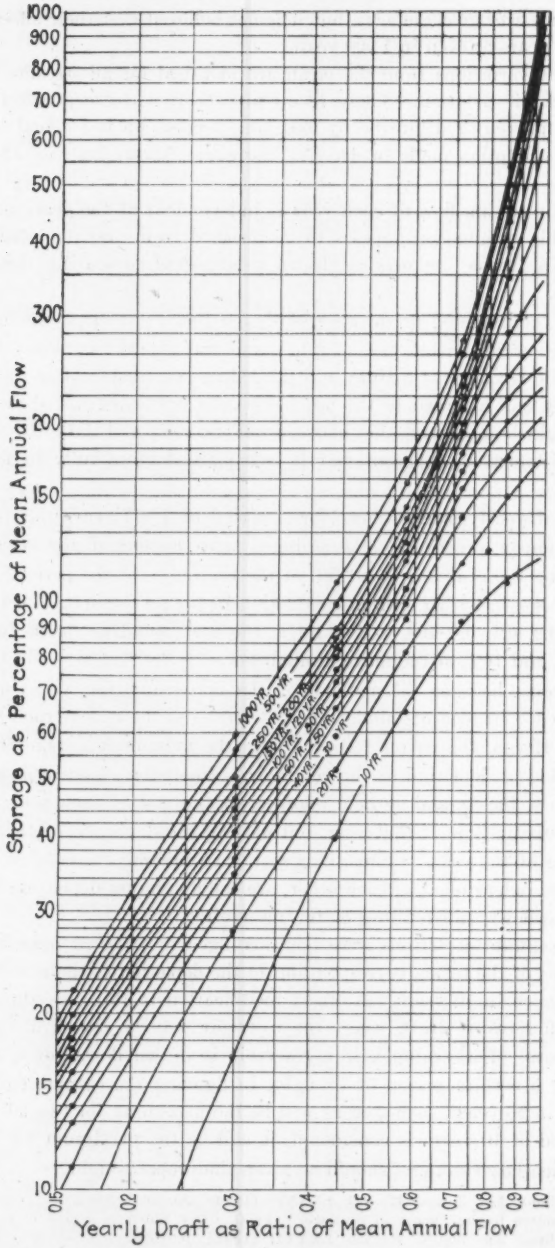
Storage as Percentage of Mean Annual Flow
1000
900
800
700
600
500
400
300
200
150
100
90
80
70
60
50
40
30
20
15
10
0.15
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
Yearly Draft as Ratio of Mean Annual Flow
1000 YR.
500 YR.
250 YR.
100 YR.
80 YR.
60 YR.
50 YR.
40 YR.
30 YR.
20 YR.
10 YR.

FIG. 14.

Orders of magnitude were converted into frequency in years by the relation:

$$\text{Frequency} = \frac{1\,000}{\text{order of magnitude}}$$

and the results plotted on logarithmic paper. Smooth curves were drawn through or near the plotted points, and in no case was the deviation of the plotted points from the smooth curves very great.

From this diagram, the annual storages required for frequencies of 10, 20, 30, 40, 50, 60, 80, 100, 120, 150, 200, 250, 500, and 1 000 years were recorded for each of the seven rates of draft, and to these quantities were added the allowance for monthly storage as given in Table 20. The final results were plotted on Fig. 14 and smooth curves were drawn through the plotted points.

Fig. 14, therefore, represents the frequency with which the storage capacity of an impounding reservoir will be equalled or exceeded for uniform rates of draft, when various proportions of the mean annual flow are diverted from the drainage area, for streams having a coefficient of variation of 0.70. The storages are expressed in percentages of the mean annual flow, so that in order to use Fig. 14 for estimating the storage required for any rate of draft on any stream having a coefficient of variation of 0.70, all that is necessary is an accurate value of the mean annual flow. If it is necessary, a correction of the allowance for monthly storage can be made; that is, if the monthly flows of the stream being investigated show that the mean monthly storage is greater or less than the normal storage chosen in Table 20, then the proper correction may be made.

In Table 21, a comparison will be made, using Fig. 14, of the frequency with which the storage required in Mr. Hazen's 95% year will probably occur for streams having a coefficient of variation of 0.70.*

TABLE 21.—Comparison of Frequency of Storage for 95% Year for Coefficient of Variation of 0.70.

Percentage of mean flow available.	Storage in terms of mean flow.*	Frequency, in years, from Fig 14.
90	4.70	150
85	3.70	150
80	3.10	180
75	2.60	200
70	2.20	250
65	1.85	300
60	1.60	350
55	1.35	350
50	1.15	400
45	0.98	400
40	0.84	450
35	0.72	500
30	0.61	1 000

Table 21 illustrates the point which the writer wished to make, by showing that in all cases the period of drought requiring the storage estimated for the 95% year will occur less frequently than once in 100 years, for the reason that when such storage is exceeded it will usually be in groups of several years.† The

* American Civil Engineers' Handbook, 1920 edition, p. 1197.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1596.

increasing infrequency of the storage required in the 95% year for lower rates of draft is due to the different method of allowing for monthly storage; and also because of the relation assumed by Mr. Hazen that the storage is proportional to the coefficient of variation does not hold strictly, and an analysis of this relation shows (for high coefficients of variation) the fallacious requirement of storage to maintain zero drafts.*

Fig. 14 has been checked against the records of a dozen or more streams having coefficients of variation of approximately 0.70, and the results in all cases were very satisfactory. Other diagrams applicable to other coefficients of variation can be made in the manner described by means of Fig. 13.

From Fig. 14 it is seen that it is only a matter of time before any nominal storage capacity which might be provided in an impounding reservoir for a uniform rate of draft will be exceeded. Limitations must be put on the storage from other considerations.

In the case of reservoirs for municipal supply, the needs for water steadily increase due to the growth of the cities; so that in designing water supply systems, it is customary to provide sufficient water to supply the probable needs for from 10 to 20 years in the future, with the knowledge that at the end of that time further expansion of the system will be necessary. Hence, if a system is designed to supply the increasing needs of a community for a period of 20 years, it is unnecessary to provide storage for the period of drought occurring only once in 1 000 years as there is only one chance in fifty that such a drought will occur during the next 20 years. Similar limitations can be placed on the size of reservoirs built to store water for irrigation or power purposes.

For reasonable safety, sufficient storage should be provided so that there is a probability of from 1:2 to 1:5 that the storage will be equalled or exceeded in the period for which the system has been designed. On this basis, the minimum storage to provide should be that equalled or exceeded in 40 years; and the maximum, that equalled or exceeded in 100 years.

Another reason for not providing too large a storage capacity is the uncertainty in the value of the mean flow. All that is known *a priori* of the observed mean is that there is one chance in two that the true mean lies somewhere within the limits set by the probable error.† To assist in determining whether the observed mean run-off is too high or too low, comparison can be made of rainfall records covering longer periods of time than are available for run-off measurements. These should be used with caution, as the rainfall stations often do not cover the drainage area and the records are frequently subject to error. Flinn states that little faith can be placed in American rainfall records previous to 1880.‡ In many large cities gauges have been moved from original stations near the ground to the tops of high buildings where the conditions affecting the collection of rain are different. The error in such records has been illustrated in the case of San Francisco.§

An example will be given to illustrate the use of Fig. 14 and to indicate the accuracy of the results obtained. Assume that the mean annual run-off of a

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1593, and Fig. 30.

† It is possible for the observed mean to vary from the true mean by more than two, three, and four times the probable error, *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1613.

‡ "Waterworks Handbook", Flinn, Weston, and Bogert, p. 3.

§ *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 526–529, and Vol. LXXXI (1917), p. 183.

stream as obtained from a 20-year record is 100 000 acre-ft. and that the coefficient of variation is 0.70 ±. The probable error of the mean would be about 11%, or there would be a probability of 1:2 that the true mean lies between 89 000 acre-ft. and 111 000 acre-ft. Assuming that a diversion of 50 000 acre-ft. per year is desired, it is necessary to allow for evaporation from the reservoir. This can be taken as 5% of the mean flow or 5 000 acre-ft., making the gross draft 55 000 acre-ft. The gross draft is then 55% of the observed mean, and probably ranges between 62% and 49% of the true mean. The storage equalled or exceeded once in 40 years and 100 years, for all these conditions, is given in Table 22.

TABLE 22.—EXAMPLE OF RANGE OF STORAGE REQUIRED TO MAINTAIN A FIXED GROSS DRAFT.

Mean annual flow, in acre-feet.	Gross draft, 55 000 acre-ft., as percentage mean flow.	Storage Equalled or Exceeded.			
		Once in 40 years.		Once in 100 years.	
		Percentage.	Acre-feet.	Percentage.	Acre-feet.
89 000	62	113	100 000	141	158 000
100 000	55	91	91 000	111	111 000
111 000	49	75	83 000	91	101 000

From Table 22 it is seen that, under the conditions assumed, if a storage of 100 000 acre-ft. was provided, the reservoir might be emptied at least once in 40 years if the true mean proved to be less than the observed mean by the amount of the probable error; or it might be emptied only at least once in 100 years if the true mean proved to be above the observed mean by the amount of the probable error. It will be noted from Table 22 that decreasing the mean has the effect (other things being equal) of decreasing the frequency with which the reservoir will be emptied. With no *a priori* evidence available, it is, therefore, on the side of safety to decrease the observed mean by at least the amount of the probable error before computing the storage. The effect of the probable error of the mean on the frequency with which a reservoir will be emptied is thus clearly demonstrated, and the futility of storage calculations which presume a high degree of accuracy is at once evident.

Mr. Harza gives an example of the flood flows of the Columbia River where the data apparently follow closely to the normal law of error. Approximate computations made from Fig. 7 show that the *C.V.* and *C.S.* for the data are 0.25 and 0.45, respectively. As the record is quite long, the coefficient of skew should be fairly reliable; its value indicates that more weight should be given the great flood of 1894, which would give a greater frequency for such flows than is indicated by Mr. Harza's diagram.

Mr. Horton indicates that the fact that the variations above the mean exceeded the variations below the mean for many forms of hydrologic data had been known for some time. The writer was aware of this, but the point he wished to make was that the correlation had not always been made between this fact and the failure of such data to follow the normal law of error. Mr. Horton offers an interesting

bibliography of probability studies which should prove to be of value to those interested in the subject.

Compliance with Mr. Horton's request for a complete exposition of the limitations of the writer's graphical method for the rectification of non-linear functions would occupy too much space, and is somewhat outside the scope of the paper, but enough will be given to indicate the principle involved. The graphical method should be used only when the equation of the curve is unknown, for when the equation is known, analytical methods can be used. In the case of the equation given by Mr. Horton:

$$I = \frac{a}{b + t}$$

this can be transposed into the form

$$t = \frac{a}{I} - b$$

By placing $\frac{1}{I} = u$, the equation becomes that of a straight line, then laying off a scale of values of u and substituting the corresponding values of I, the non-uniform scale on Fig. 8 may be obtained, which, in fact, is merely a scale of reciprocals. A more complete explanation of the method with further examples may be found in Professor Lipka's book on the subject.*

The writer cannot agree with Mr. Horton's implied statement that in investigating water resources a consideration of the rainfall-run-off relation will yield results equal or even superior to those obtained from a study of the run-off records by themselves. In the estimation of run-off from rainfall under Eastern conditions, where rainfall stations are relatively numerous and the precipitation is quite uniform over large areas, satisfactory results may be obtained; but under California conditions, where rainfall stations are more scattered and the mean annual precipitation may vary between 15 and 50 in. in a distance of 14 miles (to cite a specific case),† the problem becomes more difficult. Of course, in cases where run-off records are almost entirely lacking, recourse must be made to the rainfall in order to obtain results; but in this connection, the writer has always been impressed by the statement in Mr. Parker's book introducing the discussion of the rainfall-run-off relation,‡ which will be quoted:

"No precise solution can be given, and it will be evident that the fact that the problem assumes this particular form is in reality an indication that observations of the discharge of the stream have been neglected. Thus, in a strictly scientific sense, the fact that 85 pages are devoted to a consideration of the question is almost discreditable."

This coincides with the writer's views on the subject.

Mr. Allen's discussion illustrates an interesting application of probability methods to rainfall and other data. His criticism of the failure of the Hydraulic Probability Paper to extend to 100% does not apply if the method of plotting the data given on page 210 is used. This manner of plotting is much better than the

* "Graphical and Mechanical Computations", by Joseph Lipka.

† "Rainfall and Run-off near San Francisco, Cal.," by C. E. Grunsky, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXI (1908), pp. 496, *et seq.*, Plates LIX and LX.

‡ "Control of Water", by the late Philip A Morley Parker, M. Am. Soc. C. E., 1913 Edition, p. 171.

cumulative method where the number of observations is limited, and it will be noted that by this method 50 terms are required to reach 99% and 500 terms to reach 99.9%; however, Fig. 13 will answer any objections of this nature, as the entire probability scale is shown.

Mr. Pierce advocates the use of the median in place of the mean in dealing with stream-flow records. The median has advantage over the mean in certain cases, but these are in a large measure offset by the disadvantages, which are as follows: (1) the median is not always rigidly defined and is left more or less to the mere estimation of the observer; (2) it is more influenced by what is termed "fluctuations in sampling" than the mean; and (3) it does not lend itself readily to algebraic treatment. This last objection practically eliminates the median from any work in which theoretical considerations are necessary.

In closing, it may be stated that there is no field which offers a greater opportunity for original work than statistical mathematics and its applications, and the writer hopes that this paper and the accompanying discussions will arouse more interest in this important subject.

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Paper No. 1467

INVESTIGATION OF STRESSES IN DERRICKS

By M. C. Bland,* M. Am. Soc. C. E.

With Discussion by Messrs. G. Paaswell, Charles W. Chassaing, M. J. Patterson, and M. C. Bland.

Synopsis.

In the following paper the writer has endeavored to set down as concisely as possible the stresses in the individual members of a derrick, and their variation as the boom "picks" its load, for reaches ranging from the minimum practical reach to the maximum, or "flat boom."

The boom, being the most important member, will be treated first, and with it will be considered the load falls and the boom falls.

Next will be considered the mast, together with its stiffegs and sills. Under this heading we will first analyze a simple stiffeg derrick with a 90° opening of backstays, and afterward note one of its developments, the A-frame with tie-backs instead of stiffegs.

For the sake of brevity, little mention will be made of the details, and all members will be considered as meeting at points on their center lines.

In passing, however, the writer wishes to emphasize the desirability of reducing eccentricity to a minimum in the design of all connections, as the neglect of this precaution leads oftentimes to very high unit stresses, if not to results embarrassing to the designer.

An interesting exception to the foregoing, in the case of the boom, will be touched upon. The subject of booms is divided under two main headings: Boom Thrust Axial, and Boom Thrust Eccentric.

STRESSES IN DERRICK BOOMS.

Boom Thrust Axial.

A definite boom will be taken as an example and, for simplicity, the boom thrust will be considered as applied axially.

First, the stresses will be considered in a "flat" boom and then an expression will be given for the variation in the stresses as the boom rises. The only parts of the derrick which will be dealt with are the boom, the boom falls, and the load falls.

* Steelton, Pa.

The case of the boom is that of a strut with pin ends, with an axial thrust and also a uniformly distributed lateral load. The expression for the stress per square inch in such a member is

$$f = f' + f'' = \frac{P}{A} + \frac{M_0}{Z} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Where $M_0 = Py_0 + \frac{1}{8} w l^2$;
P = the direct thrust, in pounds;
w = weight of boom per linear inch;
A = area of boom section, in square inches;
y_0 = the middle ordinate of the elastic curve;
Z = modulus of section;
$l = b$ = length of boom, in inches.

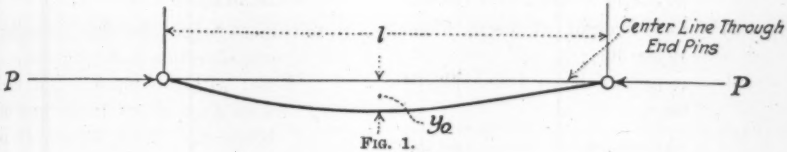


FIG. 1.

"Flat" Boom.

For this condition the lateral load causes a deflection in the strut, and the axial thrust acting with this lever arm causes a further deflection, making a total deflection of

$$y_0 = -\frac{w l^2}{8P} - \frac{w E I}{P^2}\left(1 - \sec \frac{\pi}{2}\sqrt{\frac{P}{P_e}}\right) \quad \ldots\ldots\ldots\ldots\ldots (2)$$

Where P_e = Euler's ideal collapsing load for a strut with two pin ends $= \frac{\pi^2 E I}{l^2}$;
E = modulus of elasticity.

TABLE 1.

Reach, in feet.	Sin $\theta = \frac{r}{b}$	θ	Sec θ	Remarks.
20	0.2105	12°09′	1.0229	Case IV
30	0.3158	18°24′	1.0539	"
40	0.4210	24°54′	1.1025	"
50	0.5265	31°46′	1.1762	"
60	0.6320	39°12′	1.2904	"
70	0.7365	47°26′	1.4783	"
76.44	0.8046	53°34′	1.6888	Case III
85	0.8947	63°28′	2.2385	"
90.78	0.9549	72°44′	3.3690	Case II
95	1.0000	90°00′	.....	Case I

We will next determine a suitable value for f. Using T. H. Johnson's formula for columns of mild steel, with pin ends, $\frac{l}{K} = < 159$, ultimate strength $p = 52\,500 - 220\,\frac{l}{K}$ lb. per sq. in., make $f = \frac{p}{4}$.

Under these conditions we have next to determine the load, $(L + D)$, which the boom can "pick," where L = the capacity load picked and D = one-half the boom weight, $\frac{w l}{2}$, plus W, the weight of the load falls.

Below and in Fig. 2 and Table 1 are given the boom data necessary for the computations:

Weight of 85-ft. boom = 17 323 lb.; of 10-ft. additional section = 1 905 lb.; total weight of 95-ft. boom = 19 228 lb.; or, say, $\frac{19\,228}{1\,140} = 17$ lb. per lin. in.

w = as before, weight per linear inch of boom = 17 lb.

P_e = Euler's limiting value for ideal strut, two hinged ends; *i. e.*, collapsing load $= \frac{\pi^2 E I}{l^2} = \frac{9.86 \times 29\,000\,000 \times 7\,443}{1\,299\,600} = \frac{2\,137\,000}{1.3} = 1\,645\,000$

$l = b$, length of 95-ft. boom, center to center of pins = 1 140 in.;

$w\,l$ = total weight of boom alone, = 19 300 lb.;

W = weight of boom falls, say, 6.0 tons;

$\frac{w\,l}{2}$ = half weight of boom, say, 5.0 tons;

$D = W + \frac{w\,l}{2}$ = dead load at end of boom = 11.0 tons;

A = gross area of boom section = 36.56 sq. in.

m = mast height for derrick under consideration = 56 ft. 5 in.; (*Note:* with this mast height, and with $L + D = 1.0$ at end of boom, the boom thrust, P, is 1.684;)

Z = modulus of boom section = 491 in.^{3};

I = moment of inertia = 7 440 in.^{4}

K = radius of gyration = 14.26.

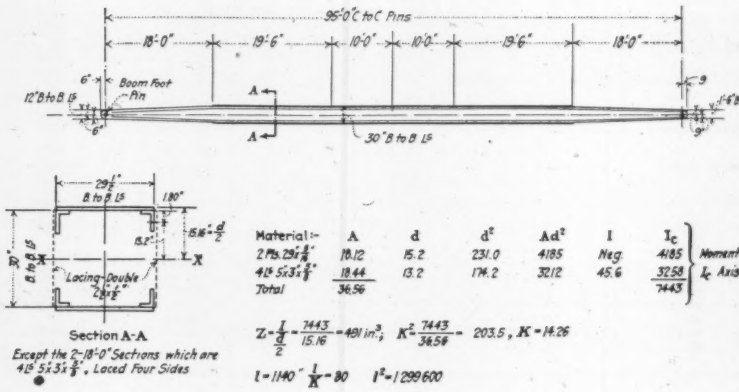


FIG. 2.

To approximate y_0, first compute the deflection of the boom under its own weight, as follows:

$$\delta = \frac{5}{384}\,\frac{W\,l^3}{E\,I}$$

W in this case being $w\,l$,

$$\delta = 0.013\,\frac{19\,300 \times 1\,481\,544\,000}{29\,000\,000 \times 7\,440} = 1.72 \text{ in.}$$

Assume that when axial thrust effect is added to the deflection due to bending under the boom's own weight, δ will be increased to 1.90 in. $= y_0$. Next, to determine the value of f, the working unit stress, find $\frac{l}{K} = 80$. Then

$$p = 52\,500 - (220 + 80) = 34\,900 \text{ lb. per sq. in.};$$

and

$$f = \frac{p}{4} = \frac{34\,900}{4} = 8\,725, \text{ say, } 8\,700.$$

Then, substituting known quantities in Equation (1), it becomes

$$8\,700 = \frac{P}{36.56} + \frac{1.9\,P}{491} + \frac{19\,300 \times 1\,140}{491 \times 8}$$

Whence

$$P = \frac{3\,100}{0.031} = 100\,000 \text{ lb.} = 1.684\,(L + D)$$

Then

$$L + D = \frac{100\,000}{1.684} = 59\,400 \text{ lb.}$$

$$D = \text{Dead Load} = W + \frac{w\,l}{2} = 22\,000 \text{ lb.}$$

$$L = \text{Capacity Load} = 37\,400 \text{ lb.} = 18.7 \text{ tons.}$$

As a check on y_0, substitute known quantities in Equation (2), which becomes

$$y_0 = -\frac{19\,300 \times 1\,140}{8 \times 100\,000} - \frac{17 \times 29\,000\,000 \times 7\,440}{10\,000\,000\,000}\left(1 - \sec 90° \sqrt{\frac{100\,000}{1\,645\,000}}\right)$$

$$= -\frac{22.0}{0.8} - \frac{36.68}{0.1}(-0.08) = -27.5 + 29.34 = 1.84 \text{ in.}$$

Hence there can be lifted a live load of 18.7 tons with a "flat" boom at a unit stress of 8 700 lb. per sq. in.

To find what part of this unit stress belongs to f' and f'', respectively, substitute the known values in Equation (1), then

$$f = f' + f'' = \frac{100\,000}{36.56} + \frac{100\,000 \times 1.84 + 2\,750\,800}{491} = 2\,735 + 5\,980 = 8\,715$$

Then $f' = 2\,735$ and $f'' = 5\,980$.

Boom Inclined.

As the boom rises, the effective span of the boom decreases proportionally to $\sin \theta$, and M_0 decreases in the same proportion, thus causing f'' to decrease correspondingly. As f is constant, 8 700 lb. per sq. in., then for a decrease in f'' there is an increase in f', and thus an increase in P, the boom thrust. That is, the boom thrust unit stress f' must increase by the same amount that f'' decreases.

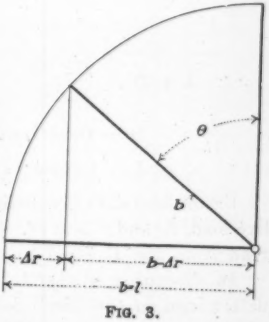


FIG. 3.

For a small increase in P:

$$P + \Delta P = A\left(f' \cdot \frac{b}{b} + f'' \cdot \frac{\Delta r}{b}\right)$$

Subtracting $P = A\,f' \cdot \frac{b}{b}$ gives $\Delta P = A\,f'' \cdot \frac{\Delta r}{b}$, that is,

$$\frac{\Delta P}{\Delta r} = A\,f'' \cdot \frac{1}{b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Now, consider the minimum practical reach, say, 20 ft.: This would be a reduction of reach of $r = 95 - 20 = 75$ ft. From Equation (3), the additional boom thrust is

$$36.56 \times 5\,980 \times \frac{75}{95} = 36.56 \times 4\,720 = 172\,500 \text{ lb.},$$

which is to be added to the original 100 000 lb., making a total of 272 500 lb. $= P$.

The new value of $f' = 2\,735 + 4\,720 = 7\,455$
and $f'' = 5\,980 - 4\,720 = 1\,260$
hence $f = 8\,715$ lb. per sq. in.

The stresses for the boom at 20-ft. reach, using the boom thrust, P, just arrived at, can be computed as follows:

First, substituting known quantities in Equation (2)

$$y_0 = -\frac{19\,300 \times 1\,140}{8 \times 272\,500} - \frac{17 \times 29\,000\,000 \times 7\,440}{74\,256\,250\,000}\left(1 - \sec 90^\circ \sqrt{\frac{272\,500}{1\,645\,000}}\right)$$

$$= -\frac{2.2}{0.218} - \frac{36.68}{0.743}(-0.245) = -10.10 + 12.10 = 2.0 \text{ in.}$$

But y_0 must be reduced by $\sin\theta = \frac{20}{95} =$ say, 0.2. Then, $y_0 \sin\theta = 2.0 \times 0.2 = 0.4$ in. The bending moment due to the weight of the boom must also be decreased in the same proportion. Then, $M_0 \sin\theta = 2\,750\,800 \times 0.2 = 550\,200$ lb. in.

Substituting these values in Equation (1)

$$f = f' + f'' = \frac{272\,500}{36.56} + \frac{(272\,500 \times 0.4) + 550\,200}{491}$$

$$= 7\,455 + 1\,340 = \text{say, } 8\,800 \text{ lb. per sq. in.}$$

That is, $f' = 7\,455$ and $f'' = 1\,340$.

Then

$$P = 272\,500 = 1.684\,(L + D)$$

$$L + D = \frac{272\,500}{1.684} = 161\,800 \text{ lb.}$$

$D =$ Dead Load $= W + \frac{w\,l}{2} = 22\,000$ lb.

$L =$ Capacity Load $= 139\,800$ lb. $= 69.9$ tons.

From these data is constructed Fig. 4, which gives straight-line formulas for the live load, L, and f' and f''. To find values of f' read up to the diagonal line, for f'' read from the diagonal line upward to the line representing f.

In designing a new boom, or investigating an existing one, the capacity is determined in the "flat" position, having first determined the mast height of the derrick so as to arrive at the boom thrust unit, in this case, 1.684. A diagram for the boom can then be constructed as in Fig. 4.

Boom Thrust Eccentric.

When the boom falls and load falls connections are not central, that is not combined at a point on the neutral axis of the boom, the boom thrust, P, is eccentric. Although these connections are symmetrically disposed with respect to this axis, as in the case of the boom now to be investigated, Fig. 5, there is only one value of θ, the angle of inclination, which gives an axial boom thrust—Case II, Fig. 5.

The worst effect of eccentricity occurs, as might be expected, in the "flat" boom. Here, P has positive eccentricity, calling eccentricity positive when it

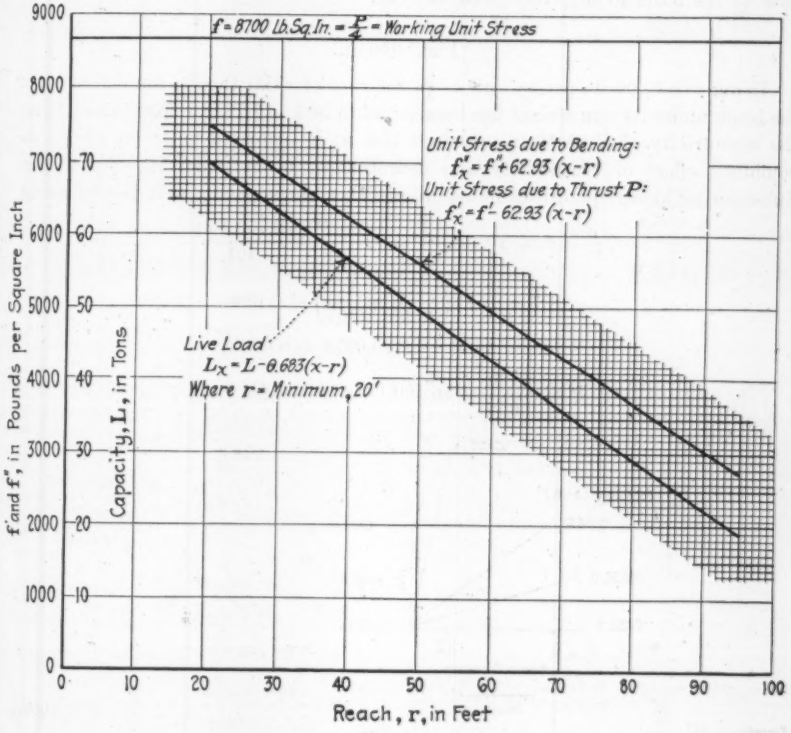


Fig. 4.

causes deflection additional to that caused by the weight of the boom itself, and negative when it has the opposite effect. For all values of θ, except for "flat" boom and Case II previously mentioned, the eccentricity of P is negative, as will be brought out in the following computations. With a constant value of the working unit stress, f, it is then evident that as θ decreases, the lifting capacity of the boom increases, not only due to the shortening of the span of the boom under bending, but also due to the negative eccentricity of P. (See Fig. 7).

The capacity of the "flat" boom, taking into consideration the eccentricity, will now be computed and compared with the same case under axial thrust. Then

the capacity of the boom for 20-ft. (minimum) reach will be determined by Equation (3). After which the capacity of the boom for this reach will be calculated and compared with the results obtained by Equation (3).

It will be found that a greater load than that given by Equation (3) can be lifted. Accordingly, a new diagram, replacing Fig. 4, can be made, and by using a straight-line formula based on the new values for "flat" boom and for minimum reach, the values of the capacity loads will be found to be higher for the same, or less, unit stress, f. For the stresses in the boom, boom falls and load falls on the basis of a load of unity, and for eccentricity of P for various reaches, see Figs. 5 and 6. For data on the boom to be investigated, see Fig. 2.

"Flat" Boom.

Using $f = 8\,700$ lb. per sq. in., as in the case of axial thrust, the deflection of the boom under its own weight has been found to be $\delta = 1.72$ in. Figs. 5 and 6 give the eccentricity of P as $4\frac{1}{8}$ in. Assume that y_0 is increased to 4.0 in. under the combined effect of eccentricity and deflection of boom under its own weight. Substituting known quantities in Equation (1), and comparing with the foregoing,

$$f = f' + f'' = \frac{P}{36.56} + \frac{4.0\,P}{491} + \frac{2\,750\,800}{491}$$

$$8\,700 = P\left(\frac{1}{36.56} + \frac{4.0}{491}\right) + 5\,600$$

$$8\,700 - 5\,600 = P\,(0.0274 + 0.0085)$$

$$P = \frac{3\,100}{0.036} = 86\,100 \text{ lb.} = 1.684\,(L + D)$$

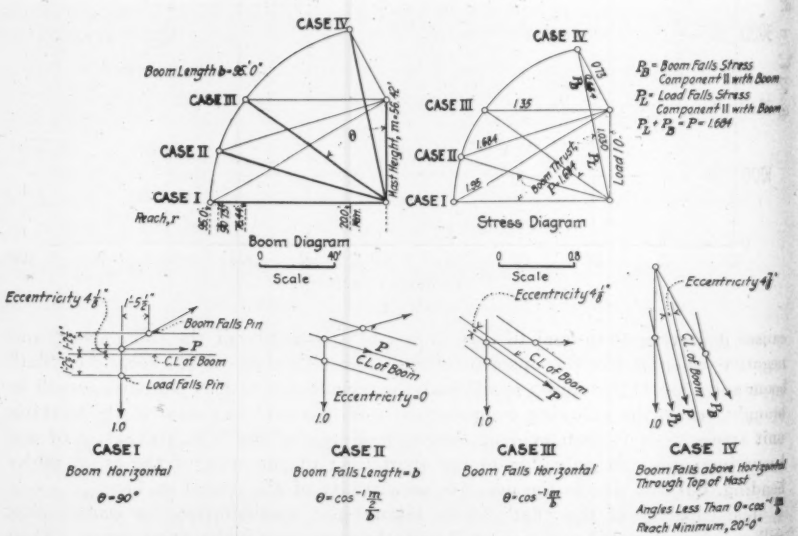


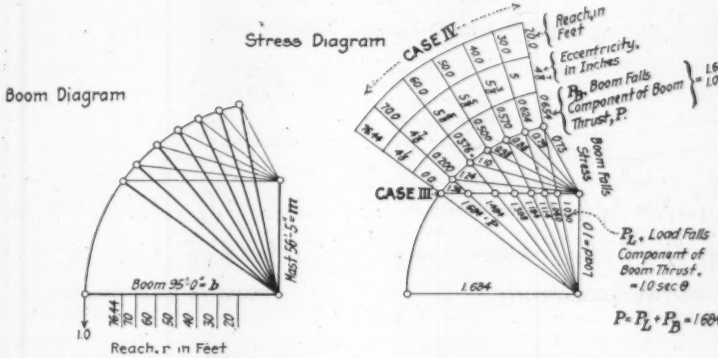
FIG. 5.

Then

$$L + D = \frac{86\,100}{1.684} = 51\,100 \text{ lb.}$$

$$D = \text{Dead Load} = W + \frac{w\,l}{2} = 22\,000 \text{ lb.}$$

$$L = \text{Live Load} = 29\,100 \text{ lb.} = 14.6 \text{ tons.}$$



CASE I – Boom Horizontal, $\theta = 90°$, $r = b$

(1) Boom Thrust, P. Taking Moments about Top of Mast:–

$+1.0 \times b - P \times m = 0 \qquad P = \frac{b}{m}$

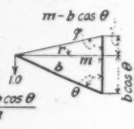
NOTE – The Higher the Mast is Made the Less will be the Resultant Boom Thrust

(2) Boom Falls Stress, $B = 1.0 \times \sqrt{\frac{b^2 + m^2}{m}}$

Eccentricity of Boom Thrust, P, is Positive, and in this case = 4⅛" – See Boom Data, Fig. 5.

CASE II Boom Falls below the Horizontal Through Top of Mast Angle greater Than $\theta = \cos^{-1}\frac{m}{b}$

In this Case the Load 1.0 is Divided into the Boom Falls and the Boom in the Proportion of $\frac{m - b\cos\theta}{m}$ and $\frac{b\cos\theta}{m}$ Respectively.



(1) Boom Stress $P = \left(1.0 \times \frac{b\cos\theta}{m}\right)\sec\theta = 1.0\,\frac{b\cos\theta}{m} \cdot \frac{1}{\cos\theta} = \frac{b\cos\theta}{m\cos\theta} = \frac{b}{m}$ as before.

(2) Boom Falls Stress $B = \left(1.0 \times \frac{m - b\cos\theta}{m}\right)\sec\varphi$ where $\varphi = \tan^{-1}\frac{r}{m - b\cos\theta}$ (NOTE Case II, Fig. 5, gives Value of θ where $b\cos\theta = \frac{m}{2}$) Eccentricity of P is Negative, for $\theta < \cos^{-1}\frac{m}{2b}$

CASE III Boom Falls Horizontal

$\theta = \cos^{-1}\frac{m}{b}$

(1) Boom Thrust, P Taking Moments about Top of Mast:–

$+1.0 \times r - P \times m \sin\theta = 0$

Then $P = \frac{r}{m\sin\theta}$, but $r = b\sin\theta$ Then

$P = \frac{b\sin\theta}{m\sin\theta} = \frac{b}{m}$ as before

(2) Load Falls Component of $P = P_L = 1.0\sec\theta = \frac{b}{m}$ and Boom Falls Component $P_B = 0$

(3) Boom Falls Stress B. Taking Moments about Foot of Mast $+1.0 \times r - B \times m = 0$, but $r = m\tan\theta$ Then

$B = \frac{m\tan\theta}{m} = 1.0\tan\theta$. Eccentricity of P is Negative.

CASE IV Boom Falls above the Horizontal Through Top of Mast. Angle Less Than $\theta = \cos^{-1}\frac{m}{b}$

(1) Boom Thrust, P. Taking Moments about Top of Mast $+1.0 \times r - P \times m \sin\theta = 0$ $P = \frac{r}{m\sin\theta} = \frac{b\sin\theta}{m\sin\theta} = \frac{b}{m}$ as before.

(2) $P_L = 1.0\sec\theta$; $P_B = P - P_L$

(3) Vertical Projection of Boom, $d - b\cos\theta$

(4) $\tan\delta = \frac{d - m}{r}$.

(5) Boom Falls Stress B (Moments at "O") $-P_B \cdot r\cos\theta + B(d - m)\cos\delta = 0$

$B = \frac{P_B \cdot r\cos\theta}{(d - m)\cos\delta}$

Eccentricity of P:– as in This Case P_B is always Less Than P_L, P is below the Neutral Axis, and is Therefore Negative.

FIG. 6.

To check the value of y_0, use the formula for positive eccentricity, Fig. 7, as follows:

$$y_0 = \left(\delta + \frac{h}{2}\right) \sec \frac{\pi}{2} \sqrt{\frac{P}{P_e}}$$

$$\sqrt{\frac{P}{P_e}} = \sqrt{\frac{86\,100}{1\,645\,000}} = \sqrt{0.052} = 0.228$$

$$\sec (90 \times 0.228) = 1.068$$

$$y_0 = (1.72 + 2.06)\,1.068 = 3.88 \times 1.068 = 4.14 \text{ in.}$$

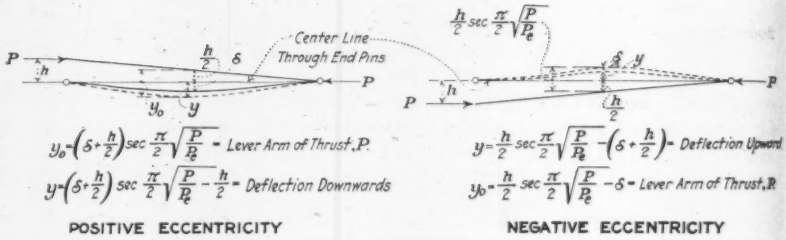


FIG. 7.

From the foregoing it is found that a live load of 14.6 tons can be lifted with "flat" boom, at a unit stress of 8 700 lb. per sq. in., whereas with axial thrust only, the boom can "pick" 18.7 tons.

To find what parts of this unit stress belong to f' and f'', respectively, substitute known values in Equation (1), using the value of y_0 just obtained:

$$f = f' + f'' = \frac{P}{A} + \frac{M_0}{Z} = \frac{86\,100}{36.56} + \frac{(86\,100 \times 4.14) + 2\,750\,800}{491}$$

$$= 2\,350 + 6\,330 = 8\,680.$$

Therefore, $f' = 2\,350$ and $f'' = 6\,330$.

Inclined Boom.

P, being applied with negative eccentricity, h, at the end of the boom, as shown in Fig. 8, has a lever arm at mid-span of

$$\frac{h}{2} \sec \frac{\pi}{2} \sqrt{\frac{P}{P_e}},$$

where $P_e = \dfrac{\pi^2 E I}{l^2}$, Euler's ideal collapsing load for the condition of two pin ends. This causes deflection of the boom upward, as represented by the dotted curved line, Fig. 8. Depending on angle of inclination, θ, part or all of this upward deflection is neutralized by the deflection of the boom under its own weight, $\delta \sin \theta$.

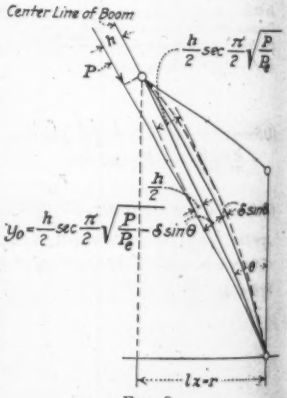


FIG. 8.

If

$$\delta \sin\theta < \frac{h}{2} \sec\frac{\pi}{2}\sqrt{\frac{P}{P_e}} - \frac{h}{2}$$

then the deflection is above the center line through the end pins and is expressed by

$$y = \frac{h}{2} \sec\frac{\pi}{2}\sqrt{\frac{P}{P_e}} - \left(\frac{h}{2} + \delta \sin\theta\right).$$

If

$$\delta \sin\theta > \frac{h}{2} \sec\frac{\pi}{2}\sqrt{\frac{P}{P_e}} - \frac{h}{2}$$

as is the case except for short reaches, then the deflection is below the center line through the end pins and is expressed by

$$y = \delta \sin\theta - \left(\frac{h}{2} \sec\frac{\pi}{2}\sqrt{\frac{P}{P_e}} - \frac{h}{2}\right).$$

For both cases the lever arm of P with respect to the center line of the deflected boom at mid-span is

$$y_0 = \frac{h}{2} \sec\frac{\pi}{2}\sqrt{\frac{P}{P_e}} - \delta \sin\theta.$$

Now, take the case of minimum value of $r = 20$ ft. From Equation (3) the additional boom thrust would be

$$36.56 \times 6\,330 \times \frac{75}{95} = 36.56 \times 5\,000 = 182\,800 \text{ lb.},$$

which added to the original $86\,100 = 268\,900$ lb. $= P$, and

$$\left.\begin{aligned} f' &= 2\,350 + 5\,000 = 7\,350 \\ f'' &= 6\,330 - 5\,000 = 1\,330 \end{aligned}\right\} f = 8\,680$$

$$L + D = \frac{268\,900}{1.684} = 159\,800 \text{ lb.}$$

$$D = \text{Dead Load} = W + \frac{w\,l}{2} = 22\,000 \text{ lb.}$$

$$L = \text{Live Load} = 137\,800 \text{ lb.} = 68.9 \text{ tons.}$$

To calculate the stresses for the aforesaid conditions, P has a negative eccentricity, h, of 4.88 in.

$$y_0 = 2.44 \sec 90\sqrt{\frac{268\,900}{1\,645\,000}} - 0.36 = (2.44 \times 1.245) - 0.36 = 2.68 \text{ in.}$$

The moment of weight of boom reduced by sin θ is

$$2\,750\,800 \times 0.21 = 577\,700 \text{ lb. in.}$$

$$P\,y_0 = 268\,900 \times 2.68 = 721\,000 \text{ lb. in.}$$

$$\text{Net moment} = 143\,300 \text{ lb. in.}$$

Then

$$f = f' + f'' = \frac{268\,900}{36.56} + \frac{143\,300}{491} = 7\,360 + 290 = 7\,650.$$

It is to be noted that the net moment, 143 300 lb. in., is negative, that is, it causes convexity upward. The amount of deflection upward from the center line through the end pins is

$$\frac{h}{2} \sec\frac{\pi}{2}\sqrt{\frac{P}{P_e}} - \left(\frac{h}{2} + \delta \sin\theta\right) = 3.04 - (2.44 + 0.36) = 0.24 \text{ in.}$$

From this it is evident that the effect of the negative eccentricity has been to decrease the bending moment, so that instead of requiring 1 330 lb. per sq. in., only 290 lb. is needed. In other words, greater live loads can be lifted and keep within the unit stress of 8 700 lb. per sq. in. The maximum load which can be lifted at $r = 20$ ft. and maintain this unit stress will now be computed.

Substituting known quantities in Equation (1) and making $y_0 = 2.75$ in. instead of 2.68 in.:

$$f = 8\,700 = \frac{P}{36.56} + \frac{(2.75\,P - 577\,700)}{491} = \frac{P}{36.56} + \frac{2.75\,P}{491} - \frac{577\,700}{491}$$

$$= P\left(\frac{1}{36.56} + \frac{2.75}{491}\right) - 1\,180$$

$$8\,700 + 1\,180 = P\,(0.0274 + 0.0056)$$

$$P = \frac{9\,880}{0.033} = \text{say, } 300\,000 \text{ lb.} = 1.684\,(L + D)$$

$$L + D = \frac{300\,000}{1\,684} = 178\,200 \text{ lb.}$$

$$D = \text{Dead Load} = W + \frac{w\,l}{2} = 22\,000 \text{ lb.}$$

$$L = \text{Live Load} = 156\,200 \text{ lb.} = 78.1 \text{ tons.}$$

Substituting known quantities to check y_0:

$$\sqrt{\frac{P}{P_e}} = \sqrt{\frac{300\,000}{1\,645\,000}} = 0.426$$

$$\sec\,(90 \times 0.426) = 1.274$$

$$2.44 \times 1.274 = 3.11 \text{ in.}$$

$$y_0 = 3.11 - \delta \sin\theta = 3.11 - 0.36 = 2.75 \text{ in.}$$

Then

$$f = f' + f'' = \frac{300\,000}{36.56} + \frac{300\,000 \times 2.75 - 577\,000}{491} = 8\,210 + \frac{247\,300}{491}$$

$$= 8\,210 + 500 = 8\,710.$$

That is, $f' = 8\,210$ and $f'' = 500$.

Then the capacity loads for "flat" boom and minimum reach, both at a unit stress of 8 700 lb. per sq. in., are known. A straight-line formula can be constructed as follows:

L varies from 78.1 tons at 20-ft. reach to 14.6 tons at 95-ft. reach, a range of 63.5 tons for 75-ft. reach variation.

Since

$$\frac{63.5}{75} = 0.847,$$

then

$$L_x = L - 0.847\,(x - r)$$

where $r =$ minimum, 20 ft.

The stress varies from 8 210 at 20-ft. reach to 2 350 at 95-ft. reach, a range of 5 860 for 75-ft. reach variation.

Since,

$$\frac{5\,860}{75} = 78.1,$$

$$f'_x = f' - 78.1\,(x - r)$$

where $r =$ minimum, 20 ft.

Value of Diagrams.

The main purpose of this paper is to show that one can avoid much unnecessary computation and keep within any assigned working unit stress, if, for any given boom (and height of mast), the capacity "flat" and at minimum reach is calculated, and a diagram constructed similar to Fig. 4 or Fig. 9. For any given reach, the live load capacity and corresponding unit stresses can then be read at a glance.

Most of the computations have been made with a 20-in. slide rule. Where, in some cases, there seems to be too much exactness in carrying out figures, as in substitutions in Equation (1) to find the boom thrust, P, this was necessary in order to get the computations to check out, as in the case mentioned, a small error in the divisor makes a large error in the quotient, P. In Fig. 6, stresses are given algebraically as well as graphically. The use of the two methods in conjunction results in a clearer view of the subject and acts to prevent material errors. A working unit stress with a safety factor of four was taken merely as an example. In many booms the stresses are run up higher, according to the judgment of the designer. It is not wise to push this too far, however, as oftentimes through misuse, improper repairs, etc., the actual conditions will be far less favorable than appear on paper.

In Fig. 9, the curved line, $X X_1 X_2$, just below the f line, represents the resulting total unit stress in the boom at various reaches. The ordinates from the f line to this curve represents the amount that the eccentricity of the boom thrust, P, decreases the bending moment in the boom. These data were calculated for the various reaches and the curve obtained therefrom.

In other words, the diagram brings out graphically the fact already mentioned, that in the case of a boom with eccentric thrust, if the maximum and minimum reaches are calculated, one will always be within the assigned value of f. This, of course, only holds true in a boom of reasonably good design. Any "freak" boom would have to be studied in detail.

Effect of Eccentricity.

In regard to eccentricity decreasing the bending moment in the boom, it is interesting to note to what extent the live load capacity is raised.

Calculations have been made for the various reaches, by assuming a value of y_0 and solving Equation (1) for the chosen value of f as follows:

$$f = 8\,700 = P\left(\frac{1}{A} \pm \frac{y_0}{Z}\right) \pm \frac{\frac{1}{8}\, w\, l^2 \sin\theta}{Z}$$

afterward correcting for y_0 if necessary.

The dotted curved line in Fig. 9 represents these new capacities, L_m, and these increased values are obtained by utilizing the part of the unit stress "left over," as represented by ordinates from the curve, $X X_1 X_2$, measured upward to the f line. The broken part of the new capacity curve, up to $r = 30$ ft., is caused by the negative bending in the boom, that is, deflection is upward. For all greater reaches the deflection is downward.

The curve, $X X_3 X_2$, Fig. 9, represents diagrammatically the percentage of increase of capacity of the boom for the corresponding reaches.

Finally, as a conservative method of making the capacity diagrams for general use in the field, it would seem best to adopt the straight-line formula as given, or one based on a somewhat higher unit stress, taking advantage of the higher capacity curve for special "picks" only.

Suggested New Arrangement.

Fig. 10 shows a suggested arrangement of load and boom falls connections, in four typical positions: Case I, flat boom; Cases II, III, and IV, minimum reach 20 ft. It will be noted that the connections are disposed so that there is no

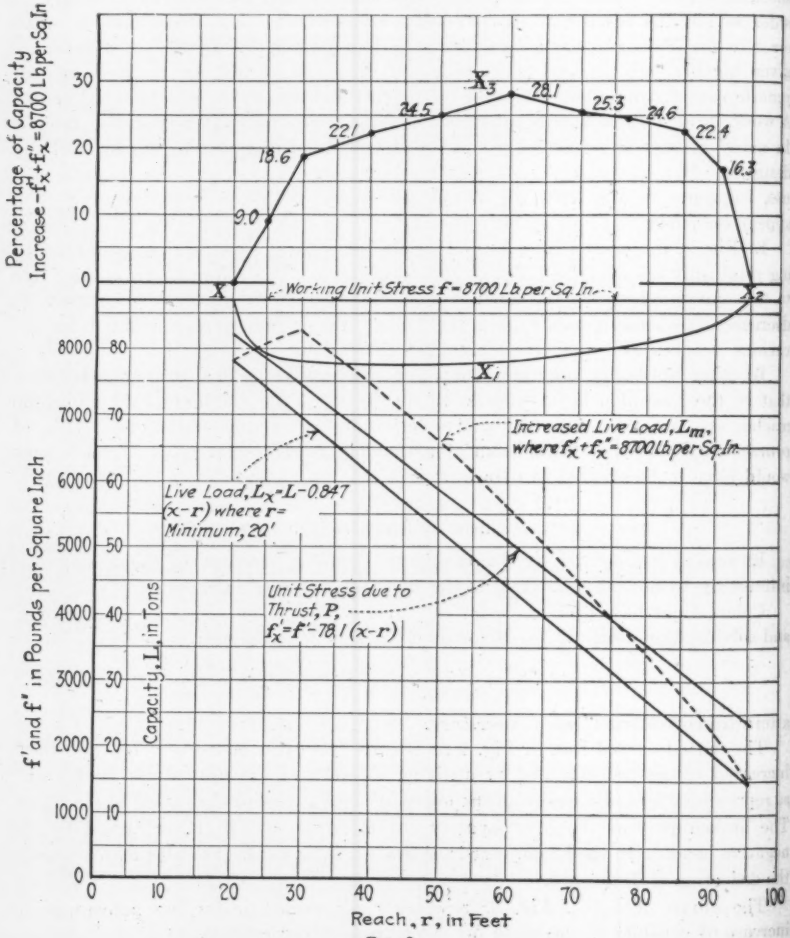


Fig. 9.

eccentricity of the boom thrust, P, for Case I. This is advantageous, as will be seen at once from the conclusions drawn after investigating the foregoing boom, where there is initial positive eccentricity of $4\frac{1}{2}$ in. On raising the boom, by this new arrangement the boom thrust, P, at once becomes negatively eccentric and continues so throughout the arc of travel.

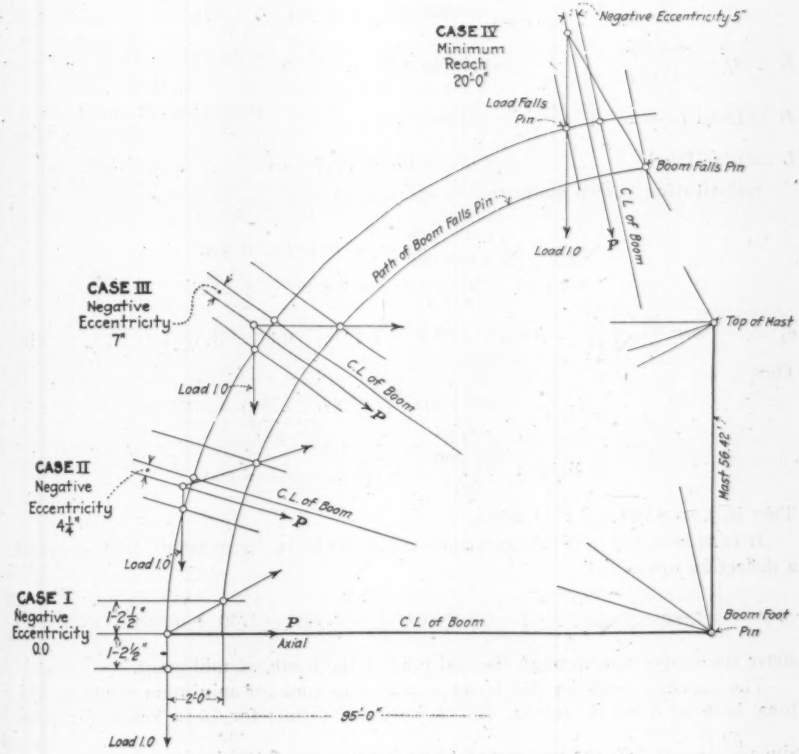


FIG. 10.

This new arrangement will now be computed for "flat" boom and minimum reach and from these data a straight-line formula will be deduced and compared with that of Fig. 9.

"Flat" Boom.

For the condition of "flat" boom, P is axial and the case is the same as under axial thrust, for which the capacity load is 18.7 tons and the unit stress, f', is 2 735 lb. per sq. in.

Boom Inclined; Case IV—Minimum Reach 20 Ft.

The eccentricity of P is 5 in., and comparing this with the similar preceding case, page 264, assume $y_0 = 2.80$ in. Substituting known quantities in Equation (1),

$$f = 8\,700 = \frac{P}{36.56} + \frac{2.80\,P - 577\,700}{491}$$

$$8\,700 + 1\,180 = P\,(0.0274 + 0.0057)$$

$$P = \frac{9\,880}{0.0331} = 298\,500 = 1.684\,(L + D)$$

$$L + D = \frac{298\,500}{1.684} = 177\,300 \text{ lb.}$$

$$D = \text{Dead Load} = W + \frac{w\,l}{2} = 22\,000 \text{ lb.}$$

$$L = \text{Live Load} = 155\,300 \text{ lb.} = 77.7 \text{ tons.}$$

$$\delta \sin\theta = 1.72 \times 0.21 = 0.36$$

Dead Load Moment $= 2\,750\,800 \times 0.21 = 577\,700$ lb. in.

Substituting known quantities to check y_0 above:

$$\sqrt{\frac{P}{P_e}} = \sqrt{\frac{298\,500}{1\,645\,000}} = \sqrt{0.182} = 0.426$$

$$\sec (90° \times 0.426) = \sec 38°\,20' = 1.274$$

$$y_0 = \frac{h}{2} \sec \frac{\pi}{2}\sqrt{\frac{P}{P_e}} - \delta \sin\theta = (2.50 \times 1.274) - 0.36 = 3.19 - 0.36 = 2.83 \text{ in.}$$

Then

$$f = f' + f'' = \frac{298\,500}{36.56} + \frac{(298\,500 \times 2.80) - 577\,700}{491}$$

$$= 8\,160 + \frac{258\,800}{491} = 8\,160 + 530 = 8\,690$$

That is, $f' = 8\,160$ and $f'' = 530$.

It is to be noted that the net moment, 258 800 lb. in., is negative, that is, causes a deflection upward of

$$y = \frac{h}{2} \sec \frac{\pi}{2}\sqrt{\frac{P}{P_e}} - \left(\frac{h}{2} + \delta \sin\theta\right) = 3.19 - (2.50 + 0.36) = 0.33 \text{ in.}$$

above the center line through the end pins of the boom, at mid-point.

The capacity load for flat boom is 18.7 tons and for minimum reach is 77.7 tons, both at 8 700 lb. per sq. in. L varies 59.0 tons for 75-ft. reach variation. Since $\frac{59.0}{75.0} = 0.787$, the new straight-line formula for capacities is

$$L_x = L - 0.787\,(x - r),$$

where r = minimum reach, 20 ft.

The stress, f', varies from 8 160 lb. per sq. in. at 20-ft. reach to 2 735 lb. per sq. in. at 95-ft. reach, a difference of 5 425 for 75-ft. reach variation.

Since $\frac{5\,425}{75} = 72.4$,

$$f'_x = f' - 72.4\,(x - r)$$

where r = minimum reach, 20 ft.

The result of this arrangement has been to raise the capacity for "flat" boom from 14.6 tons to 18.7 tons, an increase of 28%, but the capacity at minimum reach is practically unchanged, being reduced from 78.1 tons to 77.7 tons, a decrease of one-half of 1 per cent. The formula is $L_x = L - 0.787\ (x - r)$ instead of $L_x = L - 0.847\ (x - r)$, so that the capacities for all intermediate reaches are increased. This arrangement is therefore a more advantageous one.

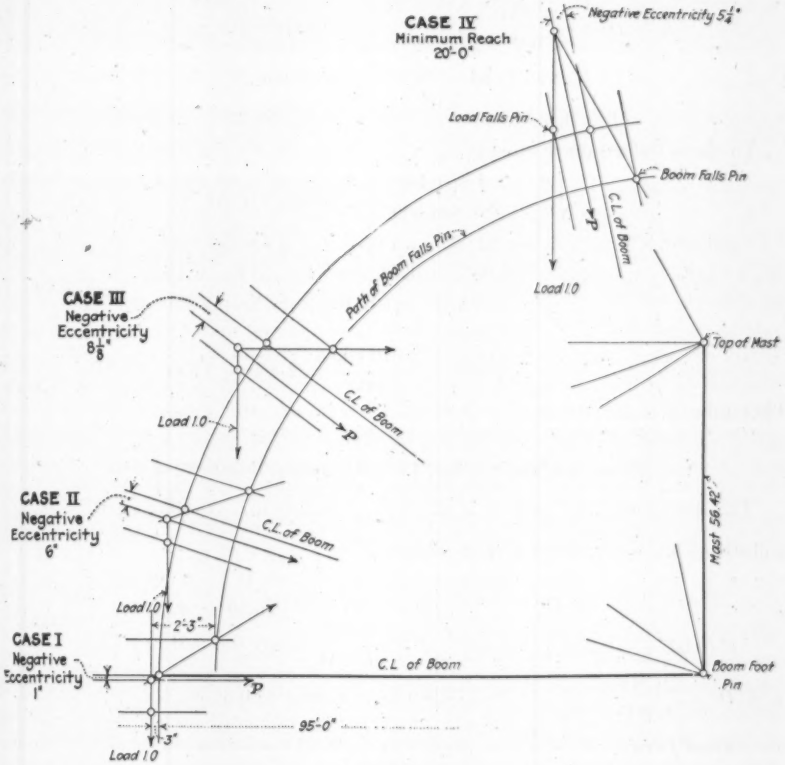


FIG. 11.

One more set of calculations will be made to see if the boom capacity can be further increased, by examining an arrangement of load and boom falls connections such as to give a slightly negative eccentricity for "flat" boom, as in Fig. 11.

"Flat" Boom.

The eccentricity is negative and equals 1 in.

From Fig. 7:

$$y_0 = \frac{h}{2} \sec \frac{\pi}{2} \sqrt{\frac{P}{P_e}} - \delta$$

Assume $y_0 = 1.15$ in., and substitute known quantities in Equation (1):

$$f = f' + f'' = \frac{P}{36.56} + \frac{1.15}{491} + \frac{2\,750\,800}{491}$$

$$8\,700 = P\left(\frac{1}{36.56} + \frac{1.15}{491}\right) + 5\,600 \qquad P\,(0.0274 + 0.0023) = 3\,100$$

$$P = \frac{3\,100}{0.0297} = 104\,400$$

$$L + D = \frac{104\,400}{1.684} = 62\,000 \text{ lb.}$$

$$D = \text{Dead Load} = W + \frac{w\,l}{2} = 22\,000 \text{ lb.}$$

$$L = \text{Live Load} = \overline{40\,000} \text{ lb.} = 20.0 \text{ tons.}$$

To check the assumed value of y_0

$$\sqrt{\frac{P}{P_e}} = \sqrt{\frac{104\,400}{1\,645\,000}} = \sqrt{0.063} = 0.251$$

$\sec (90° \times 0.251) = \sec 22° 35' = 1.083$ $\qquad \delta$ for flat boom $= 1.72$ in.

$$y_0 = (0.50 \times 1.083) - 1.72 = 0.54 - 1.72 = 1.18 \text{ in.}$$

$$f = f' + f'' = \frac{104\,400}{36.56} + \frac{(104\,400 \times 1.15) + 2\,750\,800}{491}$$

$$= 2\,860 + \frac{2\,870\,800}{491} = 2\,860 + 5\,850 = 8\,710$$

Therefore, $f' = 2\,860$ and $f'' = 5\,850$.

Boom Inclined; Case IV—Minimum Reach 20 Ft.

The eccentricity of P is now $5\frac{1}{4}$ in., and $\frac{h}{2} = 2.63$ in. Assume $y_0 = 3.0$ in.

Substitute known quantities in Equation (1):

$$f = 8\,700 = \frac{P}{36.56} + \frac{3.0\,P - 577\,700}{491} = P\left(\frac{1}{36.56} + \frac{3.00}{491}\right) - 1\,180$$

$$P\,(0.0274 + 0.0061) = 9\,880 \qquad P = \frac{9\,880}{0.0335} = 294\,800 = 1.684\,(L + D)$$

$$L + D = \frac{294\,800}{1.684} = 175\,000 \text{ lb.} \qquad \delta \sin\theta = 0.36 \text{ in.}$$

$$D = \text{Dead Load} = W + \frac{w\,l}{2} = 22\,000 \text{ lb.} \qquad \text{Dead Load Moment} = 577\,700 \text{ lb. in.}$$

$$L = \text{Live Load} = \overline{153\,000} \text{ lb.} = 76.5 \text{ tons.}$$

Substituting known quantities to check y_0:

$$\sqrt{\frac{P}{P_e}} = \sqrt{\frac{294\,800}{1\,645\,000}} = \sqrt{0.179} = 0.423 \qquad \sec (90° \times 0.423) = \sec 38° 04' = 1.270$$

$$y_0 = \frac{h}{2} \sec \frac{\pi}{2}\sqrt{\frac{P}{P_e}} - \delta \sin\theta = (2.63 \times 1.27) - 0.36 = 3.34 - 0.36 = 2.98 \text{ in.}$$

$$f = f' + f'' = \frac{294\,800}{36.56} + \frac{294\,800 \times 3.0 - 577\,700}{491}$$

$$= 8\,070 + \frac{306\,700}{491} = 8\,070 + 630 = 8\,700.$$

Therefore, $f' = 8\,070$ and $f'' = 630$.

As before, the net moment, 306 700 lb. in., is negative, that is, the deflection is upward:

$$y = \frac{h}{2} \sec \frac{\pi}{2} \sqrt{\frac{P}{P_e}} - \left(\frac{h}{2} + \delta \sin \theta\right) = 3.34 - (2.63 + 0.36) = 0.35 \text{ in.}$$

above the center line through the end pins of the boom, at mid-point.

The capacity load for "flat" boom is 20.0 tons and for minimum reach is 76.5 tons, both at 8 700 lb. per sq. in. L varies 56.5 tons for 75-ft. reach variation. Since $\frac{56.5}{75.0} = 0.754$, the straight-line formula is:

$$L_x = L - 0.754\ (x - r)$$

where r = minimum reach, 20 ft.

The stress, f', varies from 8 070 at 20-ft. reach to 2 860 at 95-ft. reach, a difference of 5 210 for 75-ft. reach variation. Since $\frac{5\ 210}{75} = 69.5$,

$$f'_x = f' - 69.5\ (x - r)$$

where r = minimum reach, 20 ft.

This last arrangement has resulted in raising the capacity of the boom from 14.6 tons to 20.0 tons, an increase of 37%, at the expense of cutting the capacity at 20-ft. reach from 78.1 tons to 76.5 tons, a decrease of 2 per cent. The cause of this decrease is naturally the increase in the negative bending moment, hence the initial negative eccentricity ("flat" boom) cannot, with economy, be made much greater.

Practically speaking, however, this "paper decrease" of capacity for minimum reach does not amount to much. The increased capacity for "flat" boom is often very advantageous, and, therefore, it would seem advisable, in the design of new booms, to try to make use of this negative eccentricity.

STRESSES IN MAST, STIFFLEGS, AND SILLS.

Simple Stiffleg Derrick.

Under this heading will be considered the stresses in the mast, stiffegs, and sills of a stiffleg derrick with 90° opening, as shown by Fig. 12. The boom is shown in three positions, Cases II, III, and IV. It is to be noted that the horizontal components of the boom falls stress and boom thrust, H_f and H_b, respectively, form a couple with arm = m, the mast height, and its tendency to rotate is resisted by the opposite couple formed by the sill stress, S_l, and the horizontal component of the stiffleg stress, H_g.

The vertical component of the boom falls stress causes compression in the mast in Case II (also Case I), zero stress in Case III, because with horizontal boom falls there is no vertical component, and tension in Case IV. The vertical component of the stiffleg stress causes compression in the mast. The stress in the mast is the algebraic sum of these two components. The vertical component of the boom thrust does not stress the mast, but passes directly to the mast support.

As the boom swings in its path, at any given radius, the stresses in the mast are not constant for that radius, as will be shown later. The stiffleg frames, in their two planes at right angles, bear varying portions of the stresses caused by the boom, depending on the position of the latter.

For instance, with the boom in the plane of the stiffleg frame, $O\,A$, Fig. 12, this frame takes 100% of the stresses, OB takes nothing. As the boom travels in its counterclockwise path the stresses in frame $O\,A$ decrease, until, upon its arriving in plane $O\,B$, they are zero. Each stiffleg frame therefore must be designed to take 100% of the stresses due to the boom falls pull and the boom thrust.

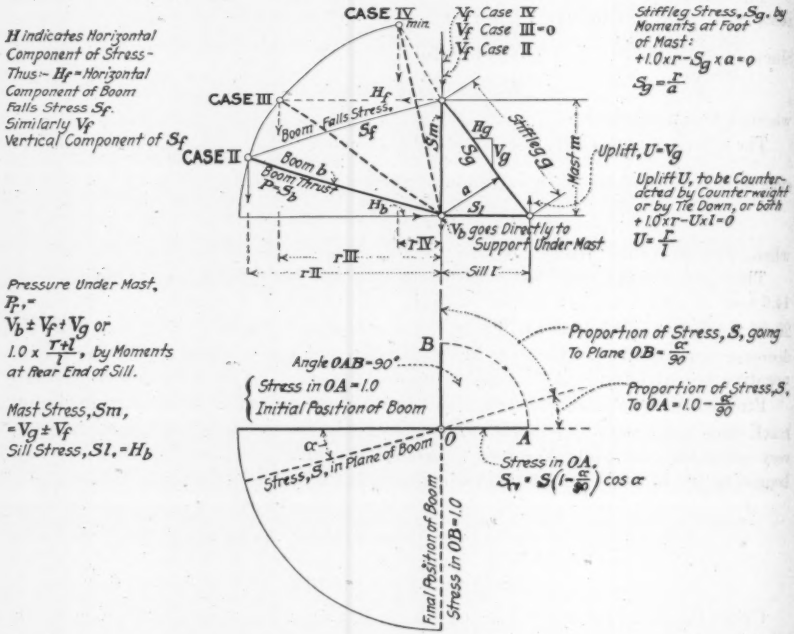


FIG. 12.

Fig. 12 shows diagrammatically the division of the stress, S, into planes, $O\,A$ and $O\,B$, for any angle, α.

Stiffleg Stresses—Case II.

For example, find the maximum stresses in such a frame for the boom at 90.73-ft. reach, as in Fig. 13. The boom falls stress and the boom thrust both equal 1.684, the horizontal component of the former stress, 1.610, is taken by the stiffleg and the vertical component, 0.50, goes to the mast, as does also the vertical component of the stiffleg stress, 2.19, making a total stress in the mast of 2.69.

The horizontal component of the boom thrust, 1.610, is taken by the sill and the vertical component, 0.50, does not affect the frame, but goes directly to the support under the mast.

At the intersection of the stiffleg and the sill, the horizontal component of the stiffleg is balanced by the sill stress, but the vertical component of the stiff-

leg stress causes an uplift, 2.19, at this point for which must be provided a "tie down," or counterweight. The dead load of the frame, etc., which can properly be considered as concentrated at this point, should be considered as counterweight, and so reduce the uplift to be taken care of.

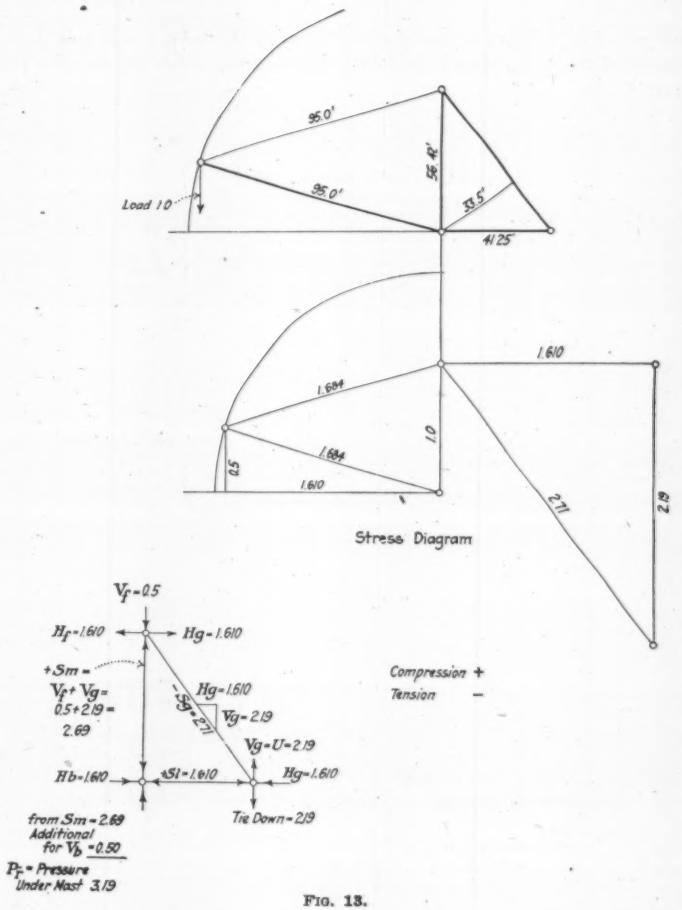


FIG. 13.

The uplift, U, due to the load, 1.0, at the boom, is found as follows:

$$1.0 \times 90.73 - 41.25\, U = 0 \qquad U = \frac{90.73}{41.05} = 2.20$$

which checks with the previous figure.

The pressure, P_r, at the support under the mast, must not be confused with the stress in the mast. With boom straight ahead in plane, $O\,A$ or $O\,B$, the pressure is found as follows:

Taking moments at point of intersection of sill and stiffleg

$$1.0 \times (90.73 + 41.25) - 41.25\ P_r = 0 \qquad P_r = \frac{131.98}{41.25} = 3.20$$

This pressure is equal to the algebraic sum of the mast stress and the vertical component of the boom thrust, or $2.69 + 0.50 = 3.19$, which checks with the value 3.20 just found.

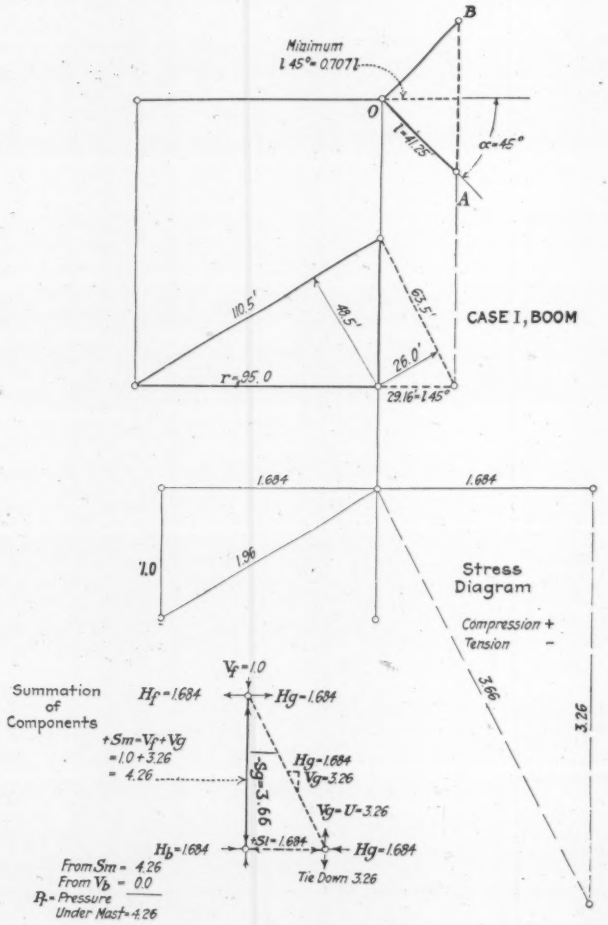
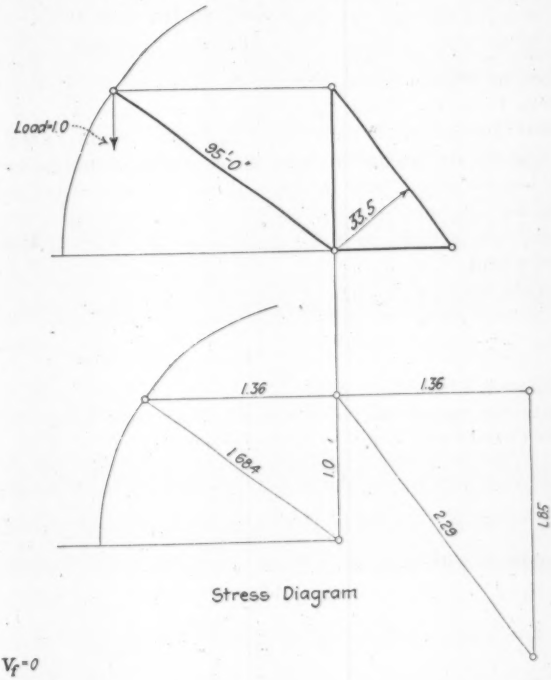


FIG. 14.

This, however, is not the maximum pressure under the mast, which is obtained as follows:

As the pressure, $P_r = 1.0\,\frac{r+l}{l}$, where r = reach and l = sill length, then, for any given value of r, the maximum value of P_r occurs when l is a minimum.



Stress Diagram

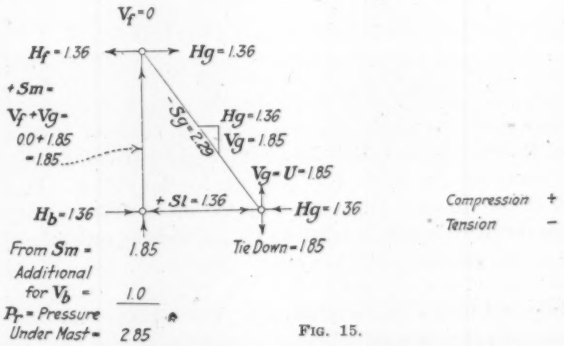


FIG. 15.

(See Fig. 14.) As the boom travels through its arc, the minimum value of l occurs when $\alpha = 45°$, and $l_\alpha = 0.707\,l$. This length varies then from $1.0\,l$, in plane of OA or $O\,B$, to $0.707\,l$ for $\alpha = 45$ degrees. Stiffleg and sill members are shown in dotted lines. The frame would be of these dimensions, and have the stresses shown, if the

two frames, *O A* and *O B*, were "folded together" to the 45° point, and the members shortened accordingly.

For any given value of reach, *r*, it has been found that the maximum value of P_r occurs when $\alpha = 45$ degrees. Then, the maximum value of P_r for the derrick is found when *r* is a maximum, that is, when the boom is "flat". For this condition:

$$P_r = 1.0 \times \frac{95.0 + 29.16}{29.16} = \frac{124.16}{29.16} = 4.26$$

Now, as already stated, Fig. 12, $P_r = V_b \pm V_f + V_g$; also, the mast stress $S_m = \pm V_f + V_g$. Then, $P_r = V_b + S_m$ and $S_m = P_r - V_b$. Therefore, to find the mast stress, compute the value of P_r for any condition and subtract V_b from it.

Evidently the maximum mast stress is then found under condition of "flat" boom, because in this case the boom thrust has no vertical component. Referring to Fig. 14,

$$S_m = \pm V_f + V_g = + 1.0 + 3.26 = 4.26$$

as just found.

For flat boom, with boom in plane, *O A* or *O B*,

$$P_r = 1.0 \times \frac{95.0 + 41.25}{41.25} = \frac{136.25}{41.25} = 3.30$$

and the mast stress, $S_m = P_r = 3.30$.

Then the mast stress, for flat boom, varies from 3.30 to 4.26.

For Case II, now being considered, boom in plane, *O A* or *O B*:

$$P_r = 1.0 \times \frac{90.73 + 41.25}{41.25} = \frac{131.98}{41.25} = 3.20$$

$$S_m = 3.20 - 0.5 = 2.70$$

For boom with $\alpha = 45°$:

$$P_r = \frac{90.73 + 29.16}{29.16} = 4.11$$

$$S_m = 4.11 - 0.5 = 3.61.$$

Then, for Case II, the mast stress varies from 2.70 to 3.61.

Stiffleg Stresses—Case III.

To find the maximum stresses in the frame for the boom at 76.44-ft. reach, Case III, see Fig. 15. The boom falls is horizontal and its stress is 1.36; there is no vertical component and the stress 1.36 is taken by the stiffleg. The vertical component of the stiffleg stress, 1.84, is taken by the mast. The horizontal component of the boom thrust, also 1.36, is taken by the sill and the vertical component, 1.0, goes directly to the support under the mast.

At the intersection of the sill and stiffleg, the horizontal component of the stiffleg stress is balanced by the stress in the sill. The vertical component of the stiffleg stress causes an uplift of 1.84.

The uplift is also figured as follows:

$$1.0 \times 76.44 - 41.25\ U = 0 \qquad U = \frac{76.44}{41.25} = 1.85$$

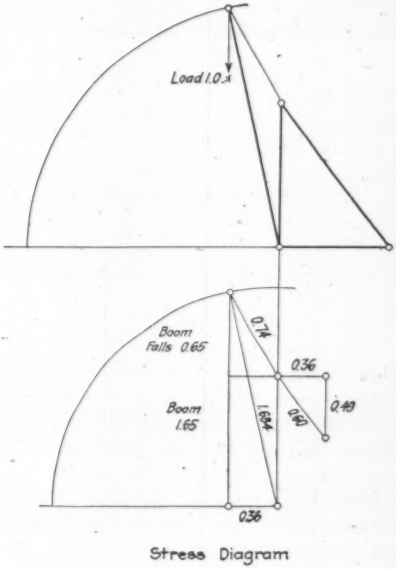
which checks.

The pressure under the mast is

$$1.0 \times (76.44 + 41.25) - 41.25\ P_r = 0 \qquad P_r = \frac{117.69}{41.25} = 2.85$$

As a check,

$$P_r = V_b + S_m = 1.0 + 1.84 = 2.84$$



Stress Diagram

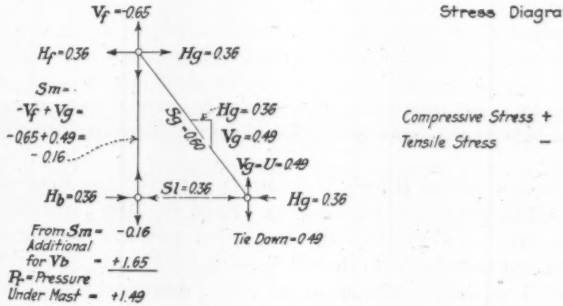


FIG. 16.

Stiffleg Stresses—Case IV.

Consider the case of minimum reach, 20 ft., under Case IV boom, Fig. 16. The boom falls stress is 0.74, its vertical component, 0.65, causes tension in the mast and its horizontal component, 0.36, causes tension of 0.60 in the stiffleg. The horizontal component of the stiffleg stress, 0.36, balances the corresponding com-

ponent of the boom falls, and its vertical component, 0.49, reduces the tension in the mast to $0.65 - 0.49 = 0.16$. The horizontal component of the boom thrust, 0.36, causes compression in the sill and its vertical component, 1.65, goes directly to the support under the mast.

At the rear connection of the sill, the horizontal component of the stiffleg stress, 0.36, is balanced by the sill stress, and its vertical component causes an uplift of 0.49.

The uplift is also figured as follows:

$$1.0 \times 20.0 - 41.25\ U = 0 \qquad U = \frac{20.0}{41.25} = 0.49$$

which checks.

The pressure under the mast is

$$1.0 \times (20.0 + 41.25) - 41.25\ P_r = 0 \qquad P_r = \frac{61.25}{41.25} = 1.49$$

As a check,

$$P_r = V_b + S_m = -0.16 + 1.65 = 1.49.$$

A-Frames.

In this development of the derrick, the **A**-frame replaces the mast. There may be two stifflegs meeting at the apex of the **A**-frame, or these may be replaced by two ties to take tension only. In this case auxiliary struts should be used to hold the **A**-frame in position, these members to run from some convenient point on the **A**-frame legs to the ground or to the sill framing, if used. In general, the construction shown on Fig. 17 will be regarded as typical. However, as in the case of the simple stiffleg derrick, this framing of course is modified to suit any local conditions.

When not supported from the ground, the vertical component of the boom thrust is sometimes sustained by a trussed sill member, as shown in dotted lines, Fig. 17. Often, however, for reasons of under-clearance, or otherwise, it is taken by a tension vertical to the head iron at the apex of the **A**-frame, whence it goes by way of the **A**-frame legs to the supports. This, of course, causes greater stresses in the **A**-frame legs.

The horizontal component of the boom thrust is borne by two diagonal sill struts and the rear sill tie takes no stress, see Figs. 17 and 18. The horizontal component of the boom falls stress is taken by the tension members meeting at the apex of the **A**-frame and transferred to the sill framing.

The "back frame" is essentially similar to the stiffleg derrick already described, except that the angle of opening of the sills is less. This will later be touched upon further.

The vertical components to be sustained by the **A**-frame are as follows, loads being given per leg:

(*a*) one-half the boom thrust;
(*b*) one-half the boom falls stress;
(*c*) one-half the tie back stress;

(*d*) in addition to these, when the boom is making a "side swing" the near leg takes an additional compressive stress caused by the side pull of the boom falls. This force also decreases, by a like amount, the compression in the far leg.

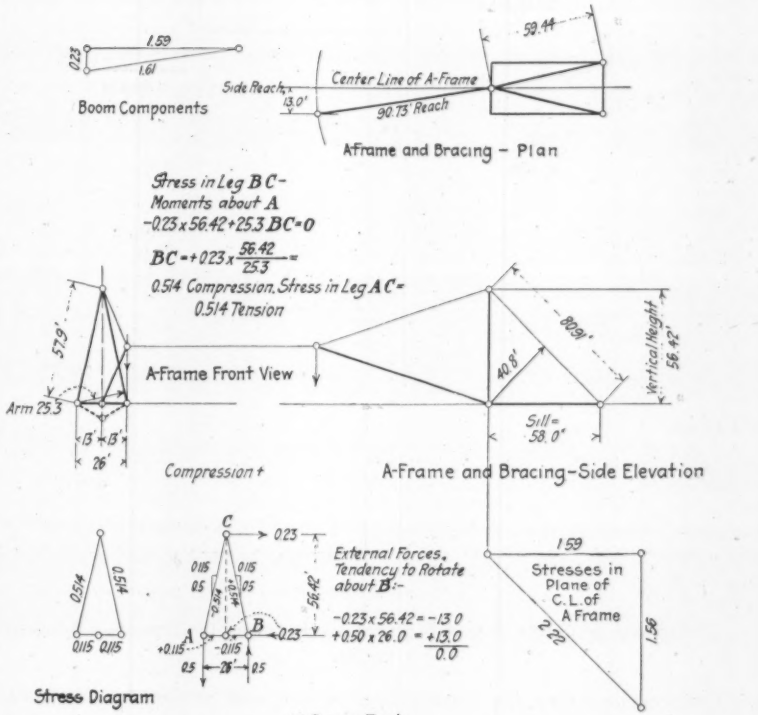
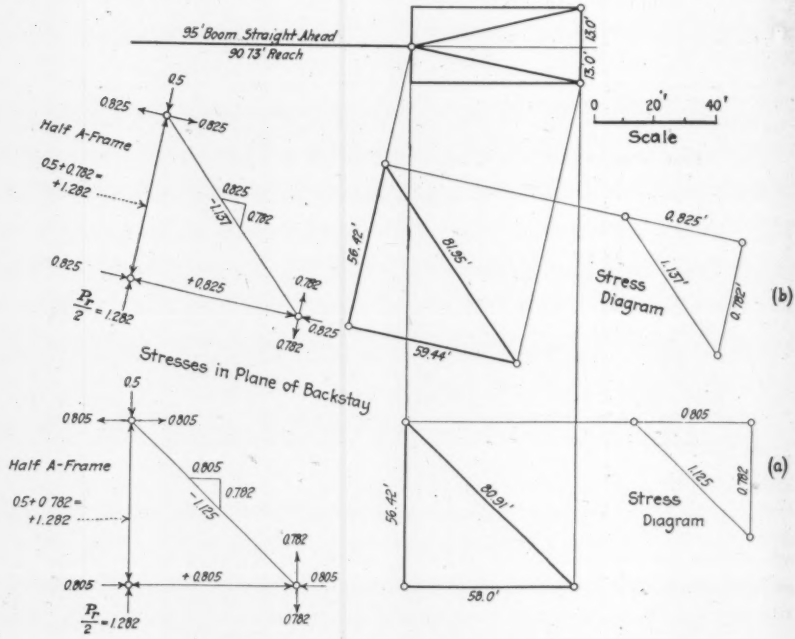


FIG. 17.

When the boom is making a side swing, the backstays running from the **A**-frame apex to the sills take the longitudinal component of the boom falls stress, and the transverse component is taken, as previously mentioned, by the **A**-frame legs.

In this case it is important to block up the **A**-frame legs firmly, otherwise there will be settlement under the near leg and unequal stresses will be transmitted to the backstays.



Stresses on Center Line of Boom for One Half Boom Thrust and Boom Falls Stress.

FIG. 18.

The pressure under the **A**-frame legs is computed as for the simple stiffleg derrick, but, in addition, when the boom is making a side swing, there is pressure under the near leg, as already mentioned, and a corresponding uplift under the far leg.

Effect of Side Swing.

Now consider the effect of the side swing on the **A**-frame:

For any side swing there are four horizontal components to be considered—the longitudinal components of the boom thrust and boom falls stress are taken care of by the boom kick struts and the backstays, respectively. The transverse horizontal components of these two stresses are the ones considered first. For any radius and side swing of boom, these two transverse components are equal and opposite, forming a negative couple with a lever arm equal to the vertical height of the **A**-frame, and the tendency is for the **A**-frame to rotate about the foot of the near leg. (See Fig. 17.)

This tendency is resisted by a positive couple with a lever arm equal to the spread of the **A**-frame legs.

Taking the example shown on Fig. 17, the reach of the boom is 90.73 ft., Case II, and the side reach is 13.0 ft. The horizontal components of the boom thrust and boom falls stress, in the plane of the boom, are each 1.610, the longitudinal component of which is 1.59, taken care of as previously mentioned. The transverse components are each 0.23 and form a couple with a lever arm of 56.42 ft., the moment of which is — 13.0.

Expressing this in another way, the load is 1.0 and the side swing 13.0 ft., hence, the moment, as before, is — 13.0. This couple has opposed to it another couple of 0.5 with a lever arm of 26.0 ft. center to center of A-frame legs, and the moment is + 13.0.

In this case there is an additional pressure under the near leg of 0.5 and an uplift under the far leg of the same amount.

The pressure under the A-frame legs for 90.73-ft. reach, with boom straight ahead, is

$$P_r = 1.0 \times \frac{90.73 + 58.0}{58.0} = \frac{148.73}{58.0} = 2.56$$

With the boom 13.0 ft. to the side, additional pressure of 0.5 under the near leg is obtained. Thus, the near leg has a pressure of

$$\frac{2.56}{2} + 0.5 = 1.78.$$

Check this by adding the vertical components coming to the A-frame legs:

	Far leg.	Near leg.
(*a*) boom thrust (see Fig. 17) each leg.....	0.25	0.25
(*b*) boom falls stress (" ") " "	0.25	0.25
(*c*) tie back stress (" ") " "	0.78	0.78
(*d*) additional, boom to side, near leg.....	...	0.50
decrease, boom to side, far leg.....	— 0.50	...
Total pressure, P_r, each leg.....	0.78	1.78
Total pressure, P_r, under A-frame.....	0.78 +	1.78=2.56

This checks with the foregoing computation.

Note the summation of the horizontal and vertical components at points *A*, *B*, and *C* on Fig. 17.

The point of application of the boom thrust is at the center of the A-frame sill. This causes compression in the far half of the sill and tension in the near half.

With 90.73-ft. reach and load 1.0 at the end of the boom, the uplift, *U*, at the intersection of tie-backs and sills, is found as follows:

Taking moments at foot of A-frame

$$1.0 \times 90.73 - 58.0\ U = 0 \qquad U = \frac{90.73}{58.0} = 1.565$$

or 0.782 per side. This uplift is at least partly counteracted by the dead load of the frame at this point, also engine, etc. The remainder, if any, must be counterweighted or tied down, or, for larger safety margin, both methods may be used.

If the reach of the boom equals the sill length, 58 ft., then the uplift $U = 1.0$; and for minimum reach, 20 ft., the uplift $= 1.0 \times \frac{20.0}{58.0} = 0.345$.

Next, consider the remainder of the stresses in the frame, for the boom at 90.73-ft. reach straight ahead (Fig. 18). The horizontal components of the boom thrust and the boom falls stress are each 1.610, or 0.805 per side of frame. Fig. 18 (*a*) gives the stress diagram and components for one side of the bracing, considering it as being on the center line of the boom. Fig. 18 (*b*) gives these stresses resolved into the plane of the backstays.

It is to be noted that the uplift, U, at the rear end of the sill, is 0.782, which checks with the foregoing uplift computation.

Throughout this paper only one value of boom thrust, $P = 1.684$, has been dealt with because it was most convenient for the purpose. This was accomplished by making the vertical height of the **A**-frame the same as the mast height in the simple stiffleg derrick, that is, 56.42 ft. As $P = \frac{b}{m}$, it follows that for any given boom length, b, a different value of P, and, therefore, different stresses, occur whenever the mast height, m, is changed.

There was taken for investigation a boom designed so as to be under a disadvantage in its "flat" position, because it has initial positive eccentricity, which simply adds to the work the boom has to do. In two suggested outlined designs it was shown how the boom is improved by making the boom thrust axial, or even negatively eccentric by a small amount in the "flat" position.

The fundamentals given previously are very simple. The field for skill and experience lies in the application of these rudiments and in the design of the details. An otherwise well designed derrick or traveler can be ruined by some detail which might seem too unimportant to waste time on; and, as mentioned previously, eccentricity in connections may cause some surprisingly high unit stresses.

In conclusion, the writer wishes to make clear that this paper is purely the result of computations made as a matter of interest. It is an extract from his personal notebook and not to be looked on as the approved practice of any company whose business it is to use such equipment. The figures were made primarily because the writer was not fortunate enough to happen on any reference work which dealt in detail with the stresses in booms and such equipment, but doubtless many engineers have made such computations before according to their own methods.

DISCUSSION

G. Paaswell,* M. Am. Soc. C. E. (by letter).—The author has presented a timely paper on a subject that should be of vital interest to all engineers. It seems rather strange that engineers will devote so much time to the preparation of a detailed plan, both in design and construction, for executing a piece of engineering work, and yet will make no effort to design the proper plant to build the piece of work in question. Although manufacturers have brought plant design to an efficient state, yet, of necessity, parts of the plant must be more or less standard, sacrificing flexibility to economy. There is no reason why the engineer cannot spend some effort in designing plant, and in outlining the design of a derrick, Mr. Bland has brought to the attention of the average engineer the practical methods effecting such designs.

There is little detailed criticism that the writer would bring to bear on the paper itself, except to state that possibly excess stress is placed on the use of column deflection formulas, showing more apparent, than real, accuracy. The use of the "Marston" formula seems to be a little far fetched for the conditions at hand and a simple expression is preferable, in which the usual method of flexure and column action can be combined, such as that given in a paper by the writer.†

It may be of interest to describe here a special piece of plant for a derrick-boat, which the writer was called on to design recently. Fig. 19 shows the general layout of the boom and back-stays and Fig. 20 shows the stress conditions for the maximum loading on the boom. The boom was called on to lift a clam-shell with a total dead weight, including contents, of about 9 000 lb. This load was doubled to take care of the dynamic effects, giving a static equivalent of 18 000 lb. The horizontal portion of the boom was then designed, as shown in Fig. 21, for a bending moment introduced by this 18 000 lb. placed at the end of the cantilever, and also by its own weight concentrated at its middle. There was also a direct load of 18 000 lb. Using the writer's formula, as given in the paper mentioned previously, the total allowable unit stress in this boom is equal to:

$$P = \frac{\text{Direct load}}{\text{Area}} + \frac{\text{Bending moment} \times \text{One-half diameter}}{\text{Moments of inertia}}$$

Noting that for the type of section used, see Fig. 25, the radius of gyration, r, is approximately 0.45 d, and introducing this value and using a unit stress in bending of 16 000 lb. per sq. in., the total area required is given by the following expression:

$$A = \frac{\text{Direct load}}{15\ 000} + \frac{\text{Bending moment, in inch pounds}}{6\ 400 \times \text{Diameter}}$$

The section to satisfy this area was found to be:

$$A = \frac{18\ 000}{16\ 000} + \frac{507\ 000 \times 12}{6\ 400 \times 51} = 19.7 \text{ sq. in., and } d = 51 \text{ in.}$$

To determine the section of the inclined portion of the boom, it is found that the maximum conditions for this portion are also given with the outrigger horizontal. The value of the stress in the boom falls was found by taking moments about the point, O, Fig. 20. With this stress known, the direct load in the boom

* New York City.

† *Engineering News*, June 26th, 1913.

was found graphically, as shown in Fig. 22. It is seen that the maximum moment in this portion of the boom occurs at the joint, *B*, Fig. 20, and the boom must carry the bending moment as given previously, and, in addition, the direct load as found in Fig. 22. To avoid a change in section of the angles used, the depth was increased to satisfy the conditions, using the following formula:

$$\text{Depth (inches)} = \frac{\text{Bending moment (in inch-pounds)}}{6\ 400 \times (\text{Net area} - \text{Area required for direct load only})}$$

In designing the back-stays, including the battered posts and the vertical post, it was found that the use of steel was preferable. The maximum load on the back-stays occurs when the boom is flat, as shown in Fig. 23, and with the load in the boom falls determined, as previously mentioned, the stresses in the batter

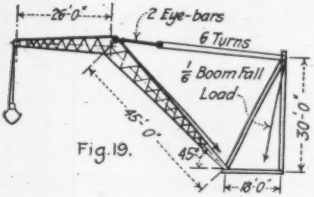


Fig. 19.

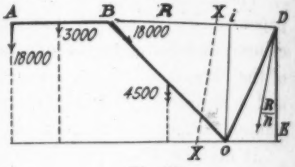


Fig. 20.

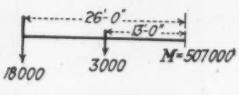


Fig. 21

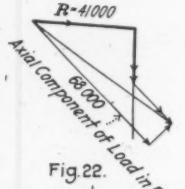


Fig. 22.

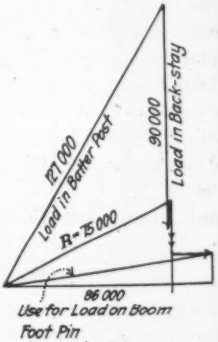


Fig. 24.

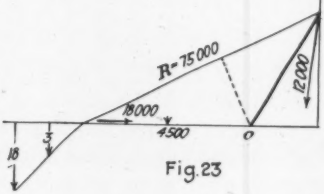


Fig. 23

post and in the vertical tension member are determined graphically as shown in Fig. 24. The complete stress and bending-moment loading, with sections designed therefor, is shown on Fig. 25. The design of the pins and pin-plates follows standard bridge practice, and need not be described here.

As a matter of general comment, it may be noted that in order to keep the boom weight as light as possible, small angles, 4 by 3 by $\frac{3}{8}$-in., were used and the depths were chosen so as to make the net section of these angles satisfy the stress requirements. For lacing, in conformity with good practice, angles were used in place of bars, and cross-frames were introduced at proper intervals to insure a well-knit column section. At the knuckle, in order to permit a proper detail for holding the boom falls, angle-bar splices were used in place of plate-splices, as

shown in detail in Fig. 25. To shorten the length of cable used, a portion of the boom falls was made up of I-bars, as shown in the detail.

The structure of course was also analyzed for side-sway, wind stresses, etc., and for a tipping of the lighter itself. It was found that the details required by the derrick base were such that ample provision was made to meet all these stresses.

An interesting feature of the design, which need not be entered into here, as not being germane to the subject-matter of the paper, was the layout of the crane base and the proper layout and arrangement of the gearing to insure maximum flexibility of movement. Although the design of the latter lies rightly within the province of the mechanical engineer, the civil engineer, of necessity, must be sufficiently familiar with gear-design theory to arrange intelligently the distribution of the plant on the crane base. The matter of proper counterbalancing to prevent any tipping of the crane, is also an interesting feature of the design.

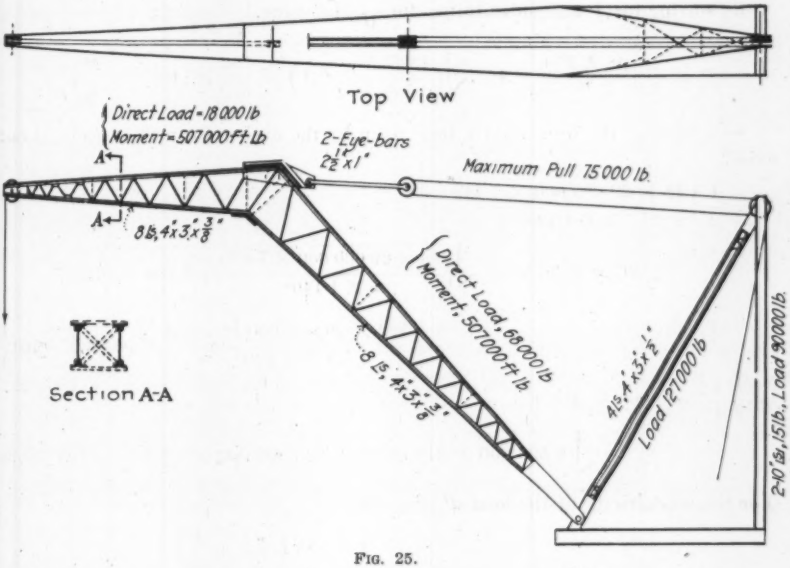


FIG. 25.

CHARLES W. CHASSAING,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read this paper with interest, and would offer a few suggestions.

By assuming the elastic curve of the neutral axis of the boom to be of the form due to a uniform load instead of the theoretical elastic curve, the value of the thrust, P, can be determined without resorting to a trial method.

Let M be the moment at mid-span of the boom for a uniformly distributed load, w. Let δ be the deflection at mid-span, due to the load, w. Let y_0 be the final deflection of the boom at mid-span, due to the load, w, plus that due to the thrust, P, acting with the lever arm, y_0.

* St. Louis, Mo.

Then,
$$\frac{y}{\delta} = \frac{M + P\,y_0}{M}, \text{ or } y_0 = \frac{\delta}{1 - \frac{P\,\delta}{M}}$$

$$\frac{\delta}{M} = \frac{\frac{5\,w\,l^4}{384\,E\,I}}{\frac{w\,l^2}{8}} = \frac{5\,l^2}{48\,E\,I} = \frac{1}{P_e}, \text{ approximately,}$$

P_e being Euler's ideal collapsing load for a strut with two pin ends.

By using the previous value of y_0, the thrust, P, can be obtained directly. The author's nomenclature will be used; M, however, will be as previously given.

$$f = \frac{P}{A} + \frac{P\,\delta}{Z\left(1 - \frac{P\,\delta}{M}\right)} + \frac{M}{Z}$$

By solving for P and substituting for $\frac{\delta}{M}$, the value, $\frac{5\,l^2}{48\,E\,I}$:

$$P = \frac{1}{2}\left\{\frac{48\,E\,I}{5\,l^2} + f\,A - \sqrt{\left(\frac{48\,E\,I}{5\,l^2} + f\,A\right)^2 - \frac{192}{5\,l^2} \times A\left(f - \frac{M}{Z}\right)}\right\}.$$

Substituting the numerical values given by the author for a "flat boom, thrust axial",

$$P = \frac{1}{2}\left\{\frac{48 \times 29\,000\,000 \times 7\,440}{5 \times 1\,140^2} + 8\,700 \times 36.56 - \sqrt{\left(\frac{48 \times 29\,000\,000 \times 7\,440}{5 \times 1\,140^2} + 8\,700 \times 36.56\right)^2 - \frac{192 \times 29\,000\,000 \times 7\,440}{5 \times 1\,140^2} \times 36.56 \times 3\,100}\right\}$$

$$= \frac{1}{2}\left\{1\,590\,000 + 318\,000 - \sqrt{(1\,590\,000 + 318\,000)^2 - 233\,000\,000 \times 3\,100}\right\} = 100\,000 \text{ lb.}$$

For the eccentricity of the load, P, Fig. 26,

$$\frac{y_1}{\frac{5\,l^2}{48\,E\,I}\left(M + \frac{P\,h}{2}\right)} = \frac{M + P\left(y_1 + \frac{h}{2}\right)}{M + \frac{P\,h}{2}},$$

$$y_0 = y_1 + \frac{h}{2} = \frac{M + \frac{P\,h}{2}}{\frac{48\,E\,I}{5\,l^2} - P} + \frac{h}{2}$$

$$f = \frac{P}{A} + \left(\frac{M + \frac{P\,h}{2}}{\frac{48\,E\,I}{5\,l^2} - P} + \frac{h}{2}\right)\frac{P}{Z} + \frac{M}{Z}$$

Solving for P,

$$P = \frac{1}{2}\left\{\frac{48\,E\,I}{5\,l^2} + f\,A + \frac{48\,E\,I\,h\,A}{10\,l^2\,Z} - \sqrt{\left(\frac{48\,E\,I}{5\,l^2} + f\,A + \frac{48\,E\,I\,h\,A}{10\,l^2\,Z}\right)^2 - \frac{192\,E\,I}{5\,l^2}A\left(f - \frac{M}{Z}\right)}\right\}$$

For negative eccentricity the terms containing h in the previous equation are minus.

Substituting the numerical values for a "flat boom, negative eccentricity 1 in."

$$P = \frac{1}{2}\left\{1\,590\,000 + 318\,000 - 59\,000 - \sqrt{(1\,590\,000 + 318\,000 - 59\,000)^2 - 233\,000\,000 \times 3\,100}\right\}$$

$$P = 104\,000 \text{ lb.}$$

These results agree closely with those obtained by the author.

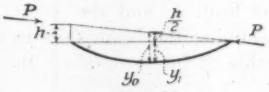


FIG. 26.

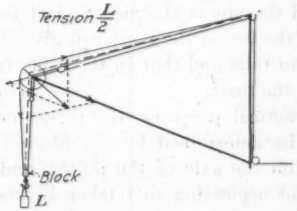


FIG. 27.

Referring to page 272, "Boom Inclined; Case IV," it seems to the writer that the greatest stress occurs at the end of the boom, to which the falls are attached, instead of at mid-span. The negative moment is $2\,948\,000 \times 5.25 = 1\,547\,700$ in-lb. The positive moment is zero. The end section of the boom is made up of four angles, 5 by 3 by $\frac{5}{8}$ in.

The sectional area is 18.44 sq. in. and the section modulus 62.4: then

$$f = f' + f'' = \frac{294\,800}{18.44} + \frac{1\,547\,700}{62.4} = 40\,300 \text{ lb. per sq. in.}$$

This would be excessive stress. At the ends of the boom a unit stress exceeding 8 700 lb. might be allowed. The writer, however, does not believe that the unit stress should exceed 12 000 lb. If a stress of 12 000 lb. is allowed, there is found

$$\frac{P}{18.44} + \frac{5.25\,P}{62.4} = 12\,000, \text{ or } P = 87\,200$$

$$L + D = \frac{87\,200}{1.684} = 53\,000$$

$$D = 22\,000$$

$$L = 31\,000 = 15.5 \text{ tons.}$$

The flat boom would carry a greater load than the inclined boom.

In regard to the consideration of eccentric thrust on the boom, the writer does not believe that it would be possible to place the load-falls pin in the position shown by the author in Fig. 11, without the cable or rope from the load falls interfering with the boom falls, as the cable or rope passes from the end of the boom to the top of the mast. The writer would propose the arrangement shown in Fig. 27. The cable of the load falls is fastened to a pin at the end of the boom by means of a yoke. The boom falls is also fastened to the pin in the same manner. The pin serves as an axle for a pulley.

Neglecting friction, the tension in the cable of the load falls will be $\frac{L}{2}$. The resultant of the tension in the cable before and after passing over the pulley will act through the axle of the pulley. This resultant will be resolved into two components, one acting along the boom, and the other acting along the boom falls.

The thrust on the boom due to the load, L, and the weight, D, can be found by drawing a right line from the top of the mast to the intersection of the resultant of the load, L, and the weight, D, with a right line drawn through the axle of the pulley and the pin at the mast end of the boom. The line drawn from this intersection to the top of the mast will give the direction of the resultant of the tension in the boom falls and that in the cable from the load falls, between the pulley and the top of the mast.

For practical purposes, the thrust on the boom and the tension in the boom falls can be determined by considering the load, L, and the weight, D, to act vertically on the axle of the pulley, and assuming the boom falls to take all the tension and neglecting that taken by the cable of the load falls. By subtracting $\frac{L}{2}$ from the tension thus found in the boom falls, the actual tension in the latter would be determined without appreciable error.

If it is desired partly to counteract the moment at mid-span of the boom due to its weight, the writer would suggest that both the axle of the pulley and the pin at the mast end of the boom be placed equally below the neutral axis of the boom. The moment at mid-span of the boom due to its weight cannot be totally counteracted without causing the stress at the ends of the boom to exceed that at mid-span. A reasonable increase in stress might be allowed at the ends of the boom over that given by the column formula.

Let h_1 be the distance that the axle of the pulley and the pin at the mast end of the boom are placed below the neutral axis of the boom. The expression for P becomes,

$$P = \frac{1}{2}\left\{\frac{48\,E\,I}{5\,l^2} + f\,A - \frac{48\,E\,I\,h_1\,A}{5\,l^2\,Z} - \sqrt{\left(\frac{48\,E\,I}{5\,l^2} + f\,A - \frac{48\,h_1\,A}{5\,l^2\,Z}\right)^2 - \frac{192\,E\,I}{5\,l^2}\,A\left(f - \frac{M}{Z}\right)}\right\}$$

This formula is not theoretically correct, as it has been assumed that the final elastic curve of the neutral axis of the boom is the elastic curve for a beam uniformly loaded. However, it gives fairly correct results for small values of h_1, and can be used to obtain preliminary values of P and y_0. The values thus

obtained can then be checked by the theoretical formula given by the author, namely,

$$y_0 = -\frac{w\,l^2}{8\,P} - \frac{E\,I\,w}{P^2}\left(1 - \sec\frac{\pi}{2}\sqrt{\frac{P}{P_e}}\right) - h_1 \sec\frac{\pi}{2}\sqrt{\frac{P}{P_e}}.$$

The writer believes that the author, in considering the stresses in the mast, has neglected to take account of the effect of the cables from the load and boom falls. These cables, as a rule, run over pulleys at the top of the mast, then down the mast to pulleys or blocks and to the drum of the hoisting engine or motor. The tension in these cables will depend on the rigging of the load and boom falls.

Although the writer has criticized some of the author's conclusions, it has been done with the best intentions. The author has covered the subject thoroughly, and the paper should be a great help to any one wishing to design a derrick.

M. J. Patterson,* Esq. (by letter).—The writer has been greatly interested in Mr. Bland's paper; not having had occasion to design any long booms, he had not before fully realized the great discrepancy in their capacity at long and at short reaches. By more familiar methods, many of the author's computations have been checked by the writer, whose particular interest has been in investigating methods for increasing the capacity at maximum reach. It seems that Mr. Bland cut off his investigation along this line prematurely.

The writer does not feel that the maintaining of a high capacity at short reach is of much importance. The only argument for this which occurs to him is that the capacity of all other parts of the derrick, including the boom line, is much greater at short reach. The writer would hesitate, however, to tell a foreman that the capacity of a derrick was 20 tons at long reach, but that he could handle 70 tons close up—he might remember the wrong figure, or he might pick up the heavy load safely and then in some emergency find it necessary to unload hastily at the nearest available spot, which might be too far away.

A fairly uniform capacity would seem much more desirable. Ordinarily, in building work, the load is picked up at long reach. If a boom is going to fail the best time for it to do so is when it is lifting the load, so possibly some excess of capacity at shorter reach may be desirable.

Mr. Bland's scheme for increasing the lifting capacity seems to work the wrong way. It gives a very small compensating moment at full reach where much is needed, and a large moment at shorter reach where less is needed.

The writer has considered the arrangement shown in Fig. 28. By varying the distance, m, the eccentricity, e, for full reach may be made anything desired. This eccentricity will always reduce to zero for vertical boom, and nearly to zero at the assumed minimum reach. Between B and C there will be a point of contra-flexure or zero moment. This may be made to occur where desired, perhaps at the center of the boom for maximum load.

Then, for maximum load, there will be no bending moment at the center of the boom at any reach, as the bending moment from the weight of the boom and from the load both vary as $\sin\theta$. For a lesser load, the point of zero moment shifts toward B, but never quite reaches it.

* Denver, Colo.

For the maximum load at maximum reach there will be a large upward bending moment at point *B*, which persists beyond this point, and only reduces to zero on reaching the center of the boom. The economy of the construction depends in part on what fiber stress may be considered safe at points between the end and the center of a strut.

Mr. Bland's investigation takes no account of conditions at or near the end of the boom. In his design, at points 18 ft. from each end, the section is reduced from 36.56 sq. in. to 18.44 sq. in. For some of the safe loads given, even with no end eccentricity, the fiber stress in these end sections will be about 17 000 lb. per sq. in. Upon applying the thrust, with eccentricity at the outer end, to the reduced section with reduced depth, excessively large fiber stresses will be found to result, and a radical change in the design of this part of the boom will be found necessary.

As involving less change of design, the writer has considered giving a boom of the type illustrated an initial camber. This seems to have the merit that the eccentricity of thrust does not increase appreciably at short reach. Comparative figures seem to show that the efficiency of a long boom at maximum reach may be increased very largely by this construction. So far as the results at the center

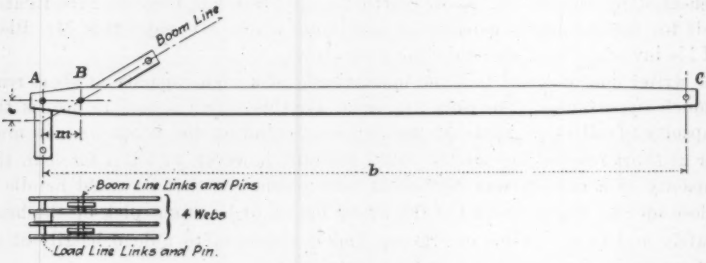


FIG. 28.

of the boom are concerned, they would be the same if the end pins are located below the center of gravity line, but the end conditions would be different in this case.

Formula for Direct Stresses in Derricks.—The writer has tabulated working formulas for direct stresses in derricks. In comparison with Mr. Bland's formulas it may be noted that there are only three dimensions and three angles involved in the construction or operation of a stiffleg derrick. Consequently, formulas for all stresses for all positions of boom can be, and preferably should be, expressed in terms of these quantities. By the use of other angles which require computation, Mr. Bland gets four boom-line formulas, although one will cover all cases. His method seems to require much figuring for special cases which need require nothing more than the application of a general formula. He really only considers special values of his angle, δ, but general formulas for any value of δ as well as of the other angles involved, are easily derived, and it is not necessary to figure stresses in any other number than the one under consideration.

About the only use, however, for the general formula is in locating the position of load for maximum stress, and in developing the maximum stress formulas, most of which are quite evident.

As the writer has found light derricks with hinged sills a great convenience in narrow bridge and viaduct work, while they are equally applicable to building work, his general formulas apply to a variable angle between the sills.

Although not strictly in the line of this discussion, the writer has found some misapprehension as to the effect of narrowing the angle between sills. The only increased stresses arising from this are, of course, those in the stifflegs and sills and in the reaction under the stifflegs, and the increase in these is less than one is apt to guess.

Table 2 shows the varying capacity of stiffleg derricks with different angles between sills, if the stifflegs and sills are only just sufficient at 90° angles. Probably 30% more material in stifflegs and sills would give the derrick full capacity when narrowed down to 45 degrees.

TABLE 2.—Comparative Capacities of Stiffleg Derricks for Different Angles Between Sills.

Angle	90°	75°	60°	45°	30°
Capacity, in tons	20	19.3	17.3	14.14	10

Frequently, an effort is made to stiffen a long boom by running the final part of the boom line from the top of the mast to the center point of the boom. This plan has some merit, if judiciously used. To gain the greatest advantage, there should be enough parts of the boom line used so that the vertical component of the strain will be about one-half of the weight of the boom.

In this case the greatest bending moment will occur half way between the outer end and the center, and will be one-fourth of the bending moment occurring in the unsupported boom at maximum reach. If, with this arrangement, the boom is raised to minimum reach position, it will then have an upward bending moment at the center, but this moment will not usually be greater than the fourth part of the bending moment occurring in the horizontal boom when unsupported at the center.

If the normal component of the strain in the boom line carried to the center is as great as the weight of the boom, the capacity of the boom at minimum reach will be lessened, and that at maximum reach will be the same as for the unsupported boom. We have simply exchanged the downward bending moment in the unsupported boom for an exactly equal upward bending moment, since a beam hung at the center has the same bending moment as when supported at the ends.

Provided, then, that the working strain in the boom line is so restricted that its normal component to the flat boom is less than the weight of the boom, there is some gain with this rig. If the normal component is not more than one-half the weight of the boom, the factor of safety will be much increased. Rather paradoxically, it is not usually possible to take advantage of this fact to increase the load very much, because to do so an impracticable number of parts of boom line must be used.

In the case of the boom specified by Mr. Bland, this scheme will raise the flat lifting capacity to about 80 tons, theoretically, but to do so requires about 25 parts of boom line.

As ordinarily used, without computation, the arrangement is dangerous, usually involving an upward bending moment much greater than the downward bending moment in the boom due to its own weight.

The writer has heretofore referred to the great reduction in the area of the end sections in Mr. Bland's design. In addition to this, the moment of inertia laterally is 4 870, as compared with 7 443, on which the unit stress is based. Neither of these conditions is compatible with a total thrust capacity of 319 000 lb. as used in the author's computations.

It seems clear that a boom should be considered as a pin-ended strut laterally, as well as vertically, and if so, the safe unit stress in this case, with the formula used, would be 7 630 lb. instead of 8 725. This would make a great difference in the results figured, as the lifting capacity is that due to the difference between the allowed fiber stress and the fiber stress from bending moment. Based on this fiber stress (7 630) the gross lifting capacity of this boom at maximum reach is 19 tons.

Six tons seems an extravagant estimate for weight of load line. Assuming 1½ tons for the weight of line, and 5 tons for the weight of half the boom, there would be left a net lifting capacity of 12.5 tons. Possibly the boom was intended to handle about 15 tons.

When one calculates the requirements in the matter of boom line and loads necessary to hold the derrick down, a 15-ton capacity with a 95-ft. boom is rather near the limit for a derrick intended to be portable. The writer is rather regretful that he did not work out examples to show how much lighter a boom could be devised to handle a load of 15 tons, rather than along the line of showing how much heavier a load could be handled with a 10-ton boom.

However the comparison may be summed up in the following approximate formula for weight of boom, B, in terms of live load, L:

$$B = C \frac{b^2 L}{m}$$

in which, B = weight of boom, in pounds;
L = live load capacity at maximum reach, in tons;
b = length of boom;
m = length of mast;
C = a constant, depending on design of boom,
= 7 for ordinary straight two-pin boom,
= 3 " boom cambered to give capacity of $\frac{5}{4}$ L at minimum reach,
= 2.6 " boom cambered to give capacity of L at both maximum and minimum reach,
= 2.4 " three-pin boom, as shown in Fig. 28.

The value of C will vary with different assumptions as to safe unit stress, but the comparison of values will be about as given.

Assuming a thrust capacity of 319 000 lb. for Mr. Bland's design of boom, the writer finds that it could be cambered to have an equal capacity at maximum and minimum reach of 66 tons gross, or 55 tons net. The necessary camber in this case is 7.4 in. With a camber of 6.08 in., it would have a gross capacity at maximum reach of about 58 tons, and at minimum reach of about 72 tons.

For this case and for most cases involving an eccentric application of direct thrust, the lifting capacity at different reaches cannot be represented by a straight-line formula. There is generally some reach at which the net bending moment becomes zero and the lifting capacity is only limited by the capacity of the boom as a strut.

Possibly some of the formulas referred to may be of interest. The one formula giving boom line stress for all cases, using Mr. Bland's notation, is:

$$\text{Boom line stress} = \frac{W}{m}\sqrt{b^2 + m^2 - 2\,l\,m\cos\theta}$$

in which W is the total load lifted.

An easier method than that used by Mr. Bland for the accurate determination of the fiber stress due to bending moment, is afforded by the Johnson-Bryan and Turneaure formula

$$f = \frac{M\,y}{I - \frac{P\,l^2}{10\,E}}$$

This formula eliminates the necessity of first guessing at, and afterward determining, the total deflection under combined stresses.

For the cambered boom, formulas for the amount of camber required to give the desired result are readily derived. For equal capacity at maximum and minimum reach:

$$Z = \frac{3\,B\,e}{40\,P}$$

where Z = required camber, in inches;
B = weight of boom, in pounds;
l = length of boom, in inches;
P = required direct thrust capacity of boom.

For this case the net bending moment at both maximum and minimum reach is $\frac{B\,l}{20}$, as compared with $\frac{B\,l}{8}$ for straight boom. The net bending moment is upward at minimum reach ($\sin\theta = 0.2$), and downward at maximum reach.

The analysis of the three-pin boom, as the writer has called it (Fig. 28), is interesting. If the desired distance of point of contraflexure, for maximum load, from inner end of boom is $n\,b$, the value of m, in inches, may be found from the equation:

$$m = \frac{12\,(1-n)\,B\,b}{2\,W + (2 - n\,B)}$$

Values of n from 0.6 to 0.07 seem to give economical results.

Since writing the foregoing, the writer has seen a boom built in this manner.

The question of the allowable fiber stress at points other than at the center of a strut, comes up again in the computation for other members of a derrick.

Practically all the formulas for combined stresses, with which the writer is familiar, are based on the maximum bending moment occurring at the center; or, at least, they do not discuss cases where much greater bending moments occur

near the ends. This is the usual case with the mast, sills, and stifflegs of a derrick. The writer would appreciate reference to a mathematical discussion of such cases.

M. C. BLAND,* M. AM. SOC. C. E. (by letter).—The writer is pleased to see that there are others who have taken an interest in the subject of derricks. This is a very broad subject, and in order to write on it with sufficient brevity it is necessary to touch very little on some features and fail to mention many others. It is an unlimited field for initiative as regards choice of general design and ingenious methods of handling details.

Just as no two pieces of erection work are exactly alike in all their conditions so they demand special plant to handle them. For example, when an erection company takes the contract for placing certain steelwork, it is foredoomed to lose money if it has a poorly conceived method of erection, or if it attempts to execute the work with improper plant. The erection engineer steadily strives to improve his plant, and in so doing he finds himself becoming not only a good structural engineer, but a mechanical and electrical engineer as well.

Referring to Mr. Paaswell's comment on the proper formula to use when designing a boom, the writer believes that in the case of long booms and other members for heavy "picks", one cannot be too careful in making the analysis of the problem at hand. The same principle governs, be a boom short or long, for light or for heavy duty, but in the latter case certain features are accentuated. Furthermore, the working conditions for a boom are different from those for a short column.

Consider a short column, Fig. 29, which in the nature of the case is more rigid than a boom. Let the column be of symmetrical section, A, and a load, P, be applied with eccentricity, e.

Then, by the usual column formula, the unit "skin" stress on the right-hand side, under the load, is

$$f = f' + f'' = \frac{P}{A} + \frac{My_1}{I},$$

where $M = P\,e$; I = moment of inertia of the section about the axis, Y-Y; and y_1 = distance from the neutral axis to the extreme fiber on the right. In this case the eccentric load increases the compression.

On the left-hand side

$$f = f' - f'' = \frac{P}{A} - \frac{My}{I}$$

y being the distance from the neutral axis to the extreme fiber on the left.

As the section is symmetrical, $y = y_1 = \frac{d}{2}$. Then:

$$f = f' \pm f'' = \frac{P}{A} \pm \frac{M}{Z}$$

which is Equation (1) of the paper.

Within practical limits, the column may be considered rigid and, as in the case of steel-cage construction, an eccentric load applied at one story may be considered axial at the story below. Other conditions obtain, however, when a boom picks its

* Steelton, Pa.

load. This the writer has endeavored to outline in his paper. For example, for the short column, $M = Pe$, whereas in the case of the boom, $M = Py_0$.

It would seem desirable to analyze this subject as strictly as possible in the first place so that one may become aware of all the factors to be considered, after which it is quite within the province of the individual designer, in any specific case, to cast aside factors which he may deem to be negligible and design the member by using any unit stress which his experience prompts.

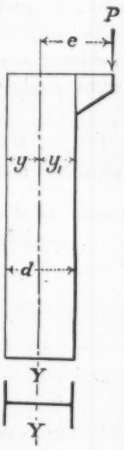


FIG. 29.

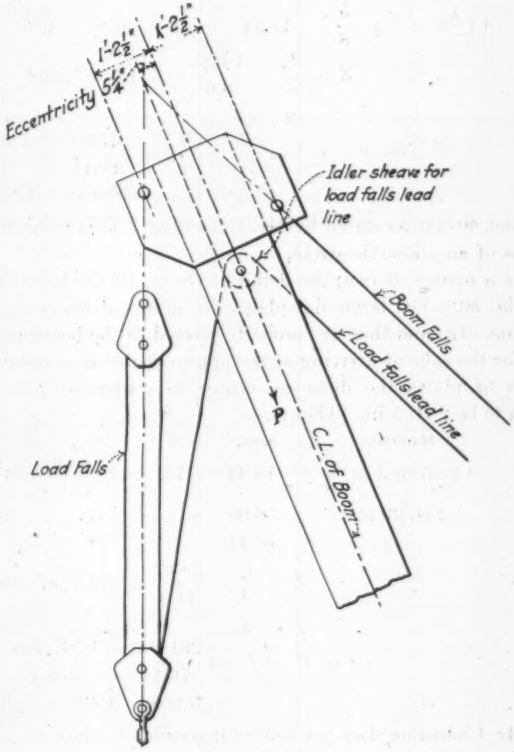


FIG. 30.

Mr. Paaswell's derrick-boat plant is very interesting, and well illustrates what the writer has just mentioned, that there is an unlimited field for initiative in meeting the every-day requirements for hoisting machinery.

With reference to Mr. Chassaing's comment on the stress at the end section of the boom which has been used as an illustration, the writer wishes to repeat what he said in the opening paragraph of his paper, namely, that for brevity's sake little mention was to be made of details at the various points. The intention was to give a bird's-eye view of the subject. The proper consideration of the many types of derricks, cranes, derrick cars, and other equipment and the development of their

many details would be a hopeless task in a short paper and would require a large volume to cover the field adequately.

If only four angles, 5 by 3 by § in. at the boom tip, are assumed to resist the bending moment due to the eccentricity of P, the result would be as follows:

Referring to Fig. 2, it is found that the end section has a depth of 18 in., back to back of angles, and the angles are placed with the long legs in the vertical plane of the boom. Then there results:

Material.	Area.	d	d^2	Ad^2	I	I_c
$4 \llcorner 5 \times 3 \times \frac{5}{8}$	18.44	7.2	51.8	956	45.6	1 002

$$Z = \frac{I_c}{\frac{d}{2}} = \frac{1\,002}{9.0} = 111.4 \text{ in.}^3, \text{ and}$$

$$f = f' + f'' = \frac{P}{A} + \frac{M_0}{Z} = \frac{294\,800}{18.44} + \frac{1\,547\,700}{111.4}$$
$$= 16\,000 + 13\,900 = 29\,900$$

and not 40 300, as stated by Mr. Chassaing. This value, however, would be far in excess of an allowable stress.

As a matter of fact, the detail at the tip of the boom in question is as outlined in Fig. 30. The large side-plates are designed heavy enough to give proper pin bearing, etc., and they are properly riveted to the boom to resist bending.

For the sake of arriving at the approximate stress obtaining, assume as effective depth of plates the distance, center to center, of pins at right angles to the boom to be 2 ft. 5 in. Then:

Material.	Area.	d	d^2	Ad^2	I	I_c
$4 \llcorner 5 \times 3 \times \frac{5}{8}$	18.44	7.2	51.8	956	45.6	1 002
2 pl. 29 × 1	58.00	....	....	....	4 065.0	4 065
	76.44					5 067

$$Z = \frac{I_c}{\frac{d}{2}} = \frac{5\,067}{14.5} = 349.4 \text{ in.}^3, \text{ and}$$

$$f = f' + f'' = \frac{294\,800}{76.44} + \frac{1\,547\,700}{349.4}$$
$$= 3\,860 + 4\,430 = 8\,290$$

Mr. Chassaing does not believe it possible to place the load-falls pin as indicated in Fig. 11 without the lead line of the load falls fouling the boom falls. This is accomplished by inserting an idler sheave, as shown in Fig. 30.

The writer has been much interested to read Mr. Chassaing's figures, and to note that the values for boom thrust agree closely with his own.

Before closing, the writer wishes to comment on the points raised by Mr. Patterson, and he will endeavor to do so in the order given.

1.—High Capacity, Short Reach versus Uniform Capacity.—Mr. Patterson seems to have building work principally in mind when he says that he does not feel that high short-reach capacity is of much importance. It is most desirable in certain cases, however, to have such high capacity; for example, in placing heavy plate girders, or when heavy bridge members are delivered to a traveler by barge or by car,

and numerous other situations that could be mentioned. The engineer has to "make the punishment fit the crime."

2.—Boom Diagrams.—The writer would not tell a foreman anything of importance about the capacity of a derrick, except in explicit cases. His recommendation in general is to furnish a diagram so that, if the foreman is of average intelligence, he can read the correct capacity for the required reach.

3.—Boom Design.—The writer deplores the fact that he did not make himself clear as to the boom which was taken by him as an illustration. He cannot agree with Mr. Patterson's statement that this boom must be "radically" changed. As President Cleveland said: "We are confronted with a condition, not a theory." This is an existing boom of mature years and faithful service. The writer did not design this boom and was in no way connected therewith. There are some points about it that he would not copy in a boom of his own. However, as a matter of fact, this boom has picked, not once but a number of times, the loads which the writer has figured it would do, in the flat position and at minimum reach.

4.—Boom-Line Attachment to Mast.—As regards making a boom-line attachment to the mid-point of the boom, the writer agrees with Mr. Patterson that it is dangerous and should be done with great caution, if at all.

5.—Unit Stresses.—Mr. Patterson calls attention to the fact that the value of I laterally is much less than the value used in the paper. He further states that the boom should be considered as a strut with two pin ends laterally as well as vertically. This is true; but in this case the value of l is much less—940 in. instead of 1 140 in.—due to the swing-line attachments which occur about 18 ft. from the boom tip. This causes the value, $\frac{l}{r}$, to be about the same as for the other axis and maintains the unit stress given in the paper.

It is to be noted that the writer did not mention the effect of wind on the boom, nor the sidewise effect of dragging a load on the ground to bring it into position to be "picked". These factors should be considered in designing a new boom. Furthermore, a wind load would cause the boom to be laterally, as well as vertically, a strut of two pin ends with end thrust, and also a uniformly distributed lateral load.

6.—Dead Load.—The writer regrets to note that, through an oversight, it was not made clear what items were included in the expression:

$$D = W + \frac{w\,l}{2}.$$

On page 259, it is stated that D = one-half the boom weight, $\frac{w\,l}{2}$, plus W, the weight of the load falls. On page 260, W is given as being "weight of boom falls, say, 6.0 tons." Mr. Patterson happened on the latter, and remarks thereon that it is an "extravagant" estimate.

The correct statement of the case is that $\frac{w\,l}{2}$ = one-half the weight of the boom, and W = weight of the load falls plus one-half the weight of the boom falls. It is to be noted that the weight of the load falls includes the weight of some heavy lifting devices, say, girder dogs, which in themselves weigh possibly a ton or more, depend-

ing on their capacity. Thanks are due Mr. Patterson for calling attention to this matter.

7.—*Choice of Formula for Combined Direct and Bending Stresses.*—Mr. Patterson's statement that the writer has made use of a formula wherein one must first guess at y_0 and afterward determine it, reminds the writer of the old bug-bear mentioned in textbooks, that in designing a column one must first assume a value of r, proportion the column, figure the value of r, and, as necessary, re-figure the column.

The writer does not consider it very difficult to arrive at a value of y_0 which would usually be about right on the first trial, provided the designer is familiar with the work in hand. This surely should not be called "guessing". As mentioned elsewhere, the individual is at liberty to choose his own formulas and unit stresses.

In closing, the writer wishes to repeat that in submitting the paper his idea was to state the case in such general terms as not to befog the fundamentals with special cases and details, nor bury them under a hopeless mass of figures. As the combined effect of direct thrust and bending is more appreciable in a moderately long, heavy-duty boom, he chose an existing boom which seemed suitable for the purpose, so that the designer of some really long heavy boom would be well aware of all the factors which should be taken into account.

As to the direct stresses in the other members of a derrick, the writer believes that the graphical method is the most expeditious and satisfactory for every-day use. The details of course are to be analyzed with as much care as the case demands. He further wishes to repeat that it is a labor-saving device to construct a capacity diagram for each boom when built, as both the erection engineer and the field forces have frequent need to refer to it.

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BANK PROTECTION AND RESTORATION: A PROBLEM IN SEDIMENTATION

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With Discussion by Messrs. D. J. Brumley, L. M. Adams, R. W. Macintyre, J. A. Ockerson, William G. Atwood, and W. C. Curd.

Synopsis.

The protection and restoration of a sliding bank along the Mississippi River at Natchez, Miss., upon which are located the tracks of the Natchez and Southern Railway, is described and illustrated in this paper. It deals with the protection of a railway track on a hill 150 ft. above low water. The hill is composed largely of sand and was subject to constant slides during low water, but of greatest extent during floods in the Mississippi River. These slides had so narrowed the space intervening between the tracks and the brow of hill at the time the work was begun that the safety of the Railway Company's traffic was in jeopardy and some means for relief had to be found. Economy was the keynote of the design, and how well this was worked out is shown by a comparison between the estimated and actual costs.

The underlying principle of the project was to produce sedimentation during floods. This was accomplished by the construction of a mud-cell dike. After the required amount of sediment had been secured, the exposed banks were permanently protected with concrete slab paving.

The completion of the work required six years, due principally to the many delays and interruptions by high water. However, Nature worked exceptionally well in the execution of the plan, which accounts for the few losses suffered.

Geological, topographical, and flood conditions at Natchez, Miss., as well as the progressive stages in the design, are described in detail. No mathematical formulas or theoretical discussions enter the paper, its chief value being a description of a practical application of sedimentation to the replacement of a river bank which had been carried away by erosion and slides.

The property and tracks of the Natchez and Southern Railway Company, at Natchez, Miss., are located for 2½ miles along the left bank of the Mississippi River.

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A succession of reverse bends above and below the city indicates the century-old changes which are taking place in the river. The Natchez bank rises precipitously to an elevation from 125 to 150 ft. above low water and being on the concave side of a sharp bend, receives the force of the current for several miles. A considerable amount of revetment has been placed by the Mississippi River Commission along the city front in past years, but it did not extend sufficiently far down stream to prevent erosion along the Natchez and Southern Company's property. However, this latter part of the bank does not receive the full effect of the current, since the channel crosses some distance above.

In 1890, when the railroad was constructed, the intervening space between the brow of the hill and the tracks was of adequate width, but protection against future loss by erosion was evidently deemed advisable, since a single row of piles, spaced from 5 to 6 ft. apart, was driven at that time along the water's edge where erosion was in progress. This work was of little benefit, and as the years went by other ineffectual forms of protection were tried with a continued loss of bank. Up to 1912, more than $50 000 had been expended in this manner without satisfactory results.

Following the severe flood of 1912, an examination of the location was made and so much of the bank was found to be lost that it was considered imperative to take steps for relief. Surveys for a new location back in the hills were made, but the estimated cost of a new line was in excess of $150 000. It was deemed advisable, therefore, to make another effort to preserve the river-front location, which could be done, it was thought, at considerably less expense than that required for a new line.

The writer, then Drainage Engineer for the Missouri Pacific System, and in charge of river-protection work, made an investigation of the conditions and designed an improvement, the completion of which was to extend over a period of from 5 to 6 years, at an estimated cost not to exceed $50 000. The design was subsequently approved and all work in connection therewith completed in the early part of 1918, at a total expense of $38 741. As the results of the work have been satisfactory, and some novel features were used, a description thereof may be of interest to the Engineering Profession.

The geologic formation of the Natchez hills consists of well-defined strata of clay, sand, and a deposit locally known as conglomerate or iron stone. The clay forms the surface covering, is very absorptive, and is 40 ft. or more in depth. The sand is clean and coarse, and lies between the clay and the conglomerate. The conglomerate outcrops slightly above low water in this location. It is dark brown in color and appears to have been deposited in a molten state. It is very hard and brittle, and where it is of any thickness satisfactorily resists erosion. Investigation showed that it lies in several layers, each separated by a light colored clay, also of a quality not easily eroded.

The sand stratum being exposed along its vertical face was continually being ground away by the action of the elements. The clay was thus undermined and, when saturated by rainfall, would break off in large pieces and slide to the foot of the bank, carrying with it large quantities of the surrounding material. Very frequently after the slides occurred the fallen materials would take a natural slope at the foot of the hill, which for a time prevented further sliding or breaking away

FIG. 1.—FORMATION OF HILL AND CONDITION AT HIGH WATER—SANDBAG PROTECTION.

FIG. 2.—MUD-CELL DIKE FROM TOP OF HILL.

at the top. However, the lapping of waves, created by passing boats, soon carried away the toe, and the slides were repeated. During high stages of the river, at a gauge of about 35 ft., direct currents would always set in and cause a rapid loss of bank.

Fig. 1 illustrates the formation of the hill and the conditions at high-water stages. Sand bags, the form of protection that was used in emergencies, are shown along water's edge. It was this condition which the writer found when he visited the location in 1912. The conglomerate was washed clean of deposits, and was exposed for varying widths beyond the foot of the hill. Parts of four short plank dikes were all that remained of the protection work previously placed, and the brow of the hill 130 ft. above was within 35 ft. of the track at the nearest point.

The previous loss of so much bank made it apparent that the correction of the difficulty lay, not only in preventing further erosion, but in a restoration of the foot of the bank so that a toe might be formed and a slope worked back to the brow of the hill. Material was scarce, as the few cuts along the track would not yield enough excavation to form the toe, and the expense of having such material loaded, hauled, unloaded, and placed in the right locations along the river-front was excessive.

Mississippi flood waters are always charged with sediment, and when the current is sufficiently retarded extensive deposits of silt can be secured. This fact, therefore, offered an economical solution of the problem of restoring the foot of the slope by taking advantage of natural conditions and producing sedimentation.

With the local conditions to be met, and having in mind his previous experience with various types of control works for producing scour and deposits, the writer decided to adopt mud-cell construction as affording about the cheapest means for getting results. The conglomerate offered a foundation secure against scour and one on which a deposit of required width and height could safely be carried. The problem then was to gather sediment from the river, deposit it to as great a height as possible, and hold it in position against erosion, so that future slides from the top of the hill would find a permanent lodging place and a stable slope be built up by casting over the side of the hill such material as would become available from time to time.

The design required that the improvement be carried out in two parts: First, to produce sedimentation, and, second, to protect permanently the slopes after sedimentation had progressed sufficiently. The first part called for the construction of a mud-cell dike with a pile dike at its head as a protection against direct currents, and the second, the construction of a low concrete toe or retaining wall with reinforced concrete slab paving on the slopes above.

In the fall of 1912, the mud-cell dike was constructed. It was 1 100 ft. long, 23 ft. high, and from 75 to 100 ft. from the foot of the hill. The distance from the foot of the hill was regulated by the width required to give a 2 to 1 slope from low water to the top of the hill. The height of 23 ft. put the top of the dike at the 37-ft. level, Natchez gauge, which was considered to be the maximum elevation to which sedimentation could be forced.

Figs. 2 and 3 are two views of the mud-cell dike, Fig. 2 having been taken from the top of the hill looking up stream. The mud cells were made from live

brush and poles which were cut and towed from an island several miles up stream from the work. A 12-in. foundation mattress, 30 ft. wide, was first laid directly on the conglomerate. This was formed by laying the brush in bundles, or fascines, which were wrapped at 4-ft. intervals with single lashings of No. 9 galvanized wire. Anchor lines of ½-in. galvanized strand, spaced 50 ft. apart and woven into the mattress, were run back to deadmen at the foot of the hill. A tier of cells, 5 by 5 ft., was built on the foundation mattress by placing single rows of brush and poles at right angles to each other, very much after the fashion of a log cabin. At a height of 5 ft., the cells were floored over with a layer of brush and a second tier of cells was started 5 ft. back from the river face of the dike. Five tiers of cells were placed in this manner, forming a completed dike 28 ft., or five cells, wide at the bottom and 5 ft., or one cell, wide at the top, and a settled height of 23 ft. All cell walls were bound with wire and cable to the foundation course, as well as to deadmen at the foot of the hill. To prevent the cells from floating when submerged, they were ballasted with loose conglomerate picked up along the river-front. A connection with the hill at the lower end of the dike was made by joining the latter to an old abandoned transfer barge which had been sunk in line during high water just previous to the commencement of the dike.

Six lateral mud-cell dikes, or baffles, 15 ft. wide by 100 ft. long, were connected into the main dike and run up the hill to Gauge 40 to prevent parallel currents and eddies in the enclosed area where sedimentation was being sought.

Through various delays in getting materials and proper pile-driving equipment, the pile dike for the head protection could not be finished with the mud cells, and an unexpected flood came on. An emergency dike was started as soon as there was a sufficient depth of water to get a marine driver on the job, but a satisfactory closure could not be made owing to difficulty in handling the driver in the heavy current. The emergency dike was breached, which resulted in the tearing loose of three uncompleted upper tiers of the main dike for a length of 100 ft. These tiers were floated inshore and, after the flood had receded, were again used by cutting the binding wires and cables and relaying them in the original position.

Fig. 4 shows the gap in the mud-cell dike caused by the breaching of the emergency dike. It also shows the extent of the deposits that were secured from one high water, even under the unfavorable conditions created by lack of head protection.

During the low-water period of 1913 the permanent pile dike to protect the mud-cell dike against head-water was completed. This dike extended from a point up stream to an outside lap connection with the mud-cell dike, a distance of 600 ft. The structure ranges from 8 to 42 ft. in height above the natural ground surface, the top being at an elevation of 56 ft. on the Natchez gauge. It is made up in width of 112 lin. ft. with piles in single rows, 216 lin. ft. in double rows, and 272 lin. ft. in three rows, with 3½ by 10-in. white oak sheeting, spaced 2 in. apart, applied to the outer, or river, face of the dike. Cypress piles ranging in length from 32 to 62 ft. were used and 16 to 20-ft. penetrations were obtained through the conglomerate, under hard driving. The piles were spaced 8 ft., center to center, and driven in staggered position so as to gain addi-

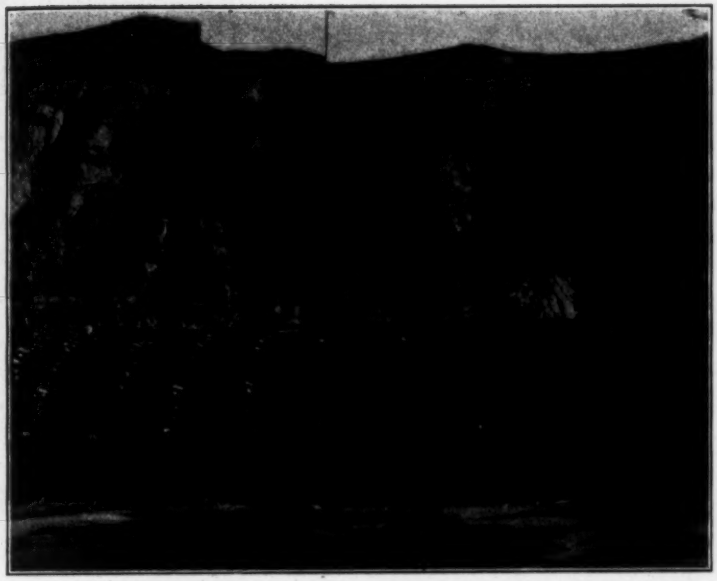


Fig. 3.—Mud-Cell Dike Showing Type of Construction.

Fig. 4.—Gap in Mud-Cell Dike Due to Break in Emergency Dike.

FIG. 5.—MUD-CELL DIKE PARTLY SUBMERGED.

FIG. 6.—COMPLETED DIKES SEEN FROM TOP OF HILL LOOKING UP STREAM.

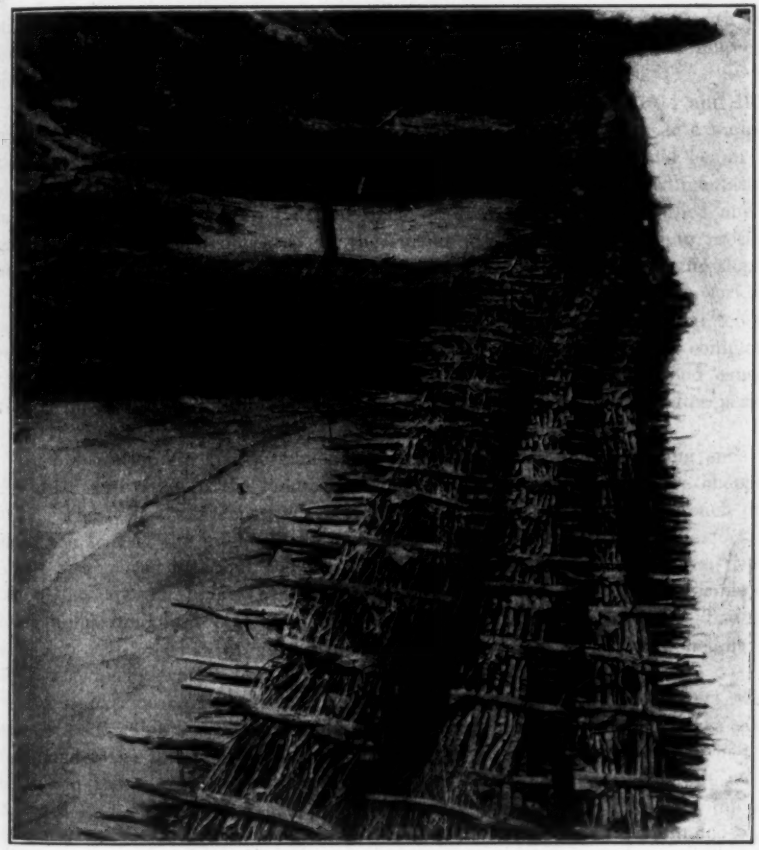
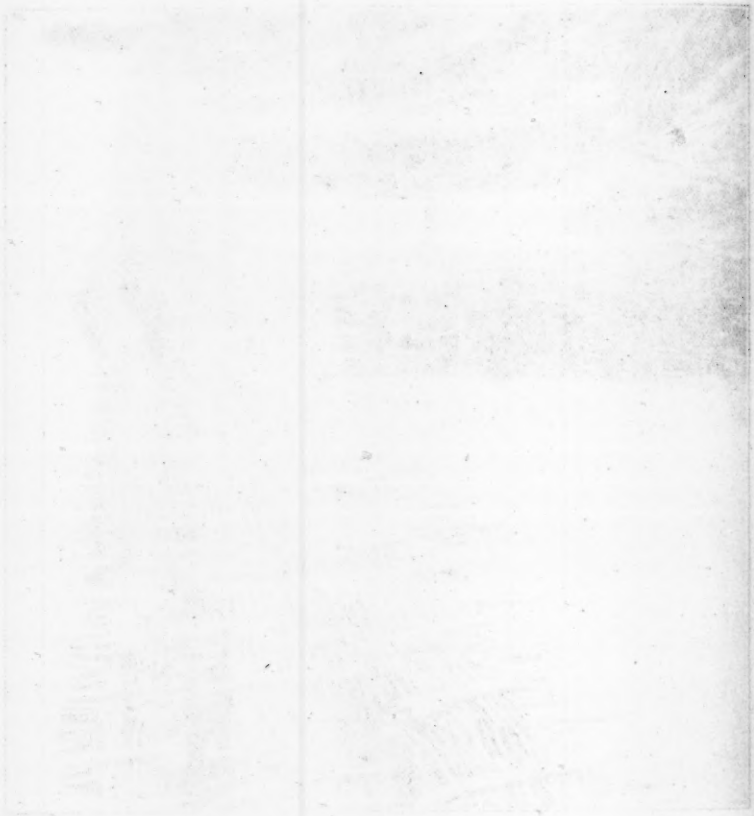


FIG. 7.—VIEW OF MUD-CELL DIKE SHOWING CONSTRUCTION DETAILS.



tional strength against overturning. This necessitated an intricate system of bracing.

The deflection of head-water in succeeding floods was successfully obtained with the pile dike after its completion in the spring of 1914, and sedimentation progressed very satisfactorily within the area enclosed by the mud-cell dike.

Fig. 5 is a view of the mud-cell dike partly submerged by high water. Fig. 6 was taken from the top of the hill looking up stream and shows the completed pile dike, mud-cell dike, and the sediment deposited after one high water, with the complete head protection. Fig. 7 is a close-up view of a part of the mud-cell dike and shows somewhat in detail the construction of the mud cells.

The dikes were permitted to stand through the high water of 1914 and 1915 and until the low water of 1916 without additions or changes. When an examination was made in the summer of 1916, it was found that decay had begun to show among the brush and poles forming the mud cells, and as sedimentation had progressed to an extent that the enclosed area was practically filled and a growth of willows had appeared on the surface, it seemed advisable to proceed at once with the second part of the plan, the permanent protection of the slopes.

Authority for the required expenditure could not be obtained until the working season of 1916 had become well advanced, but the construction of the toe or retaining wall was started in November of that year and was practically complete, together with a short section of the concrete slabs, when another flood caused suspension of the work and nothing further was done until the low-water period of 1917. The high water, however, caused no damage to the work.

In preparation for the permanent slope protection, all the projecting ends of the brush and poles in the mud-cell dike not covered by deposits, were chopped away. The débris was deposited on the slopes for further use and a trench dug along the foot of the mud-cell dike for the toe wall. The bottom of the footing course was placed at about the 10-ft. level, Natchez gauge, which gave a depth below the exposed surface of the conglomerate of from 3 to 5 ft., this depth being ample to afford protection against under-scour in this location. The toe wall was reinforced with galvanized mesh and carried to a gauge height of 14 ft., at which elevation the concrete slabs were connected to it.

When excavating the trench for the toe wall many small springs were encountered, which necessitated shoring and delayed progress. These springs came from under the hill and yielded such a quantity of water that openings had to be left in the toe wall to provide for discharge into the river. The water was collected and delivered to the openings by 6 by 6-in. drain boxes laid on top of the inside projection of the footing course. The drains were surrounded with bank-run gravel to filter the water and prevent loss of the material behind the wall.

The bulk of the sediment recovered by the mud cells was deposited between the dike and the foot of the hill, and did not completely fill the cells. After removing the projecting cells, it was found that some filling would be required to bring the surface of the slope to the elevation at which concrete paving was to be placed. This condition was provided for by casting the slabs in place on supported forms. The slabs were 4 in. thick, 54 in. wide, 14 ft. long, and reinforced with galvanized mesh. They were placed on a 3 to 1 slope, and cast in alternate sections.

On all supported work the space underneath the forms was filled with débris from the mud cells. This was done to collect sediment during high water. As the first row of slabs was nearing completion, high water came and, as was expected, sufficient sediment was collected to fill solidly the space thus left. Figs. 8 and 9 show the toe wall, the method of supporting the slab forms, and the forms complete with the reinforcement in place ready for pouring the concrete. The flooring shown in Fig. 9 was made up in panels or sections which were removed after the concrete had hardened sufficiently to support itself. Formwork was unnecessary for the upper part of the slope, and the slabs were cast directly on the ground. The amount of paving constructed was 39 744 sq. ft., being 46 ft. wide on the slope, or up to a level of 33 ft., Natchez gauge, and 864 ft. long. Figs. 10 and 11 show the concrete paving nearing completion in 1917, which was the second and last step under the plan outlined in 1912.

A subdivision of the total cost of $38 741 between the three classes of work performed is as follows:

Mud-cell dike	$9 292
Permanent pile dike	10 604
Concrete paving and wall	18 845
Total	$38 741

Cross-sections of the amount of deposits obtained through the use of the mud-cell dike were taken, but the records have been lost or cannot be located at this time. However, it is the writer's recollection that approximately 56 000 cu. yd. of material were recovered. On that basis the unit cost of restoring the foot of the bank was 16.6 cents per cu. yd., very much less than would have been required in that location by any other method. It must be borne in mind, in this connection, that had material from the hills been hauled in, some form of protection against its loss by high water would have been necessary until it had settled and the permanent paving was placed.

The general results of the work have been satisfactory and all that could have been expected. A permanent toe has been furnished on which a proper slope can be founded, and the deficiency in the width of the brow of the hill can be filled out whenever the exigencies of the situation demand it.

The frequent interruptions to construction operations caused by high water were aggravating, at times, and occasioned higher costs on some of the work than was desired, yet when the entire project is viewed as a whole, it would seem that Nature at all times worked hand in hand with the engineers.

All work, with the exception of the permanent pile dike, was executed by the Railway Company forces under various men, as assigned. Special credit is due J. C. Bauer, Inspector of River Protection, Missouri Pacific System, for the execution of the mud-cell dike; it was through his efforts, while in charge of that part of the work, that the greatest obstacle was overcome. Credit is also due Mr. C. B. Colpitts, now of Coverdale and Colpitts, New York City, for the successful completion of the pile dike. The dike was contracted for, but when about 75% complete, the contractors refused to proceed further, and it was completed by Railway Company forces working under Mr. Colpitts' direction.

FIG. 8.—METHOD OF SUPPORTING SLAB FORMS AT TOE WALL.

FIG. 9.—SLAB FORMS COMPLETE, WITH REINFORCING IN PLACE, AT TOE WALL.

FIG. 10.—CONCRETE PAVING NEARING COMPLETION IN 1917.

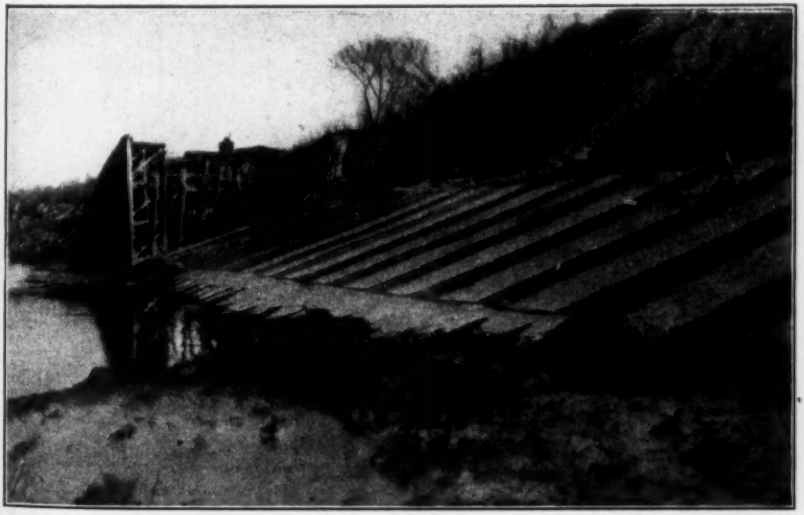


FIG. 11.—COMPLETED SECTION OF CONCRETE PAVING.

The writer left the employ of the Missouri Pacific System in the spring of 1917 during the interruption of the work on the concrete paving due to high water, and the final stage was carried to completion under the direction of his successor, J. A. Lahmer, M. Am. Soc. C. E., Principal Assistant Engineer, Missouri Pacific System.

The permanency of the completed work depends on the permanency of the head protection. When this fails the bank will go back to the condition found in 1912. The writer many times urged the local officials of the Natchez and Southern Railway Company to have the city garbage dumped behind the pile dike, so that a permanent bank might be formed by the time the life of the dike had been reached, and thus avoid the necessity and expense of rebuilding the structure. It is understood that as yet this part of the programme has not been put into effect.

DISCUSSION

D. J. BRUMLEY,* M. AM. SOC. C. E. (by letter).—Mr. Curd's development of a method for the protection and restoration of a sliding bank along the Mississippi River below Natchez, Miss., resulted from years of experience in drainage and river protection work. It is based on an intimate knowledge of the geological formation of the lower Mississippi Basin and the behavior of delta streams with reference to direction of current, hydraulic gradient, silt in suspension, and stability of banks of large streams. The development is a departure from those most generally used, and has fully demonstrated the soundness of design and the prognosis of the resultant action of stream eddy and silt deposit. It has served the purpose of more costly designs such as pile dikes filled with stone, or concrete walls.

Some provision should probably have been made to facilitate repairing the cell wall when the timber decays. This might have been provided by driving a double row of piles at the toe of the cell wall, parallel to the line of mud-cell construction. The piles would have served as a foundation for concrete paving or similar protection work whenever the results of erosion become evident.

L. M. ADAMS,† ESQ. (by letter).—The author's interesting description can be read with profit by any engineer who has to contend with the exasperating problem of an eroding bank. The Natchez and Southern Railway Company was indeed fortunate to find a hard conglomerate base on which to build the cellular dike. Unfortunately for many others who have had to solve similar problems, investigation showed no firm foundation to be available.

There is a bank-protection problem of this sort under way on the Rio Grande at Fort Brown, Texas, in the Galveston District. The bank has been rapidly caving at a bend opposite Post Headquarters until the back wall of this masonry building was almost undermined. Orders were received to protect the bank from further caving. At first it was supposed that a system of timber groins built out from the bank would stop such caving by causing slack-water between them, and hence a substantial deposit of the heavy sediment of this muddy stream. Such expectation proved to be a fallacy because of the nature of the bank.

The whole river valley is itself a sedimentary deposit of this same river during previous centuries. The Rio Grande meanders through this plain, changing its course from time to time by cutting through at bends. It is trying to do this at Fort Brown, and the Army engineers hope to prevent it. The bank, at mean low water, stands at a slope of about 1:1, and it is almost 20 ft. to the top of the bank. When the river rises, as it frequently does, it impregnates the bank to a considerable depth. When it falls, this wetted material no longer will take its previous loading and tends to flatten its slope. At some points the dead weight of the bank is sufficient to cause the river bed to bulge up at a point a few yards out from the toe of the slope. This bulge soon washes away down stream and, with the removal of this weight, equilibrium is again destroyed, with more caving and sinking of the near-by bank.

The remedy has been found to be to protect, by weighted brush mattresses shown in Figs. 12 and 13, not only the caving bank, but also to extend this erosion

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FIG. 12.—BANK AT FORT BROWN, TEXAS, PROTECTED BY WEIGHTED MATTRESSES.

FIG. 13.—WEAVING A MATTRESS SECTION, FORT BROWN, TEXAS, FOR BANK PROTECTION.

protection well out along the river bed from the toe of the bank. Some hundreds of feet of this protection work are already in place, and it seems to be solving the difficulty.

R. W. MACINTYRE,* M. AM. SOC. C. E. (by letter).—The most important feature outstanding in the author's able description of his bank protection work on the Mississippi River, would appear to be as follows: That the material for the preliminary foundation work (brush and poles) was furnished by Nature, convenient to the work, and that the river current was utilized to build up a fill, all at a very moderate expenditure.

In the writer's opinion, based on an experience of more than fifteen years, involving careful study of river-protection problems, the Engineering Profession in general appears to be slow in grasping, or making practical application of, the reconstructive usefulness to which the destructive forces of a river may be converted, both inexpensively and efficiently. To cite a definite instance, the following description of protection work carried out during the past two years in British Columbia may be of interest.

The Capilano River, with a drainage area of 64 sq. miles, rises among the lofty peaks of a mountainous peninsula lying directly north of Vancouver, B. C., Canada, at a head-water altitude of between 1 500 and 1 600 ft., thence following south for 20 miles to the mouth, where it discharges into Burrard Inlet, an arm of the Pacific Ocean. This river is the source of supply for the Vancouver water-works, the intake of which is about 7 miles up stream from the mouth, at an elevation of 480 ft., the pipe line being carried submerged across Burrard Inlet to the south shore, on which Vancouver is situated. The Pacific Great Eastern Railway crosses the Capilano River about ¼ mile from its mouth, and early in 1918 it was decided to replace the existing trestle bridge with two 100-ft. through Howe truss spans, at the same time raising the rail level 10 ft. above the original elevation, in order to eliminate the trouble caused by flood discharge carrying down trees, logs, and driftwood, which had hitherto periodically swept away the trestle bridge during high water.

The main problem to be solved in connection with the new bridge and grade was how to control the river which had widened its channel to about 900 ft. at the bridge site, although the normal width of the water surface, when confined in one channel and clear enough for fishing with rod and line, would probably not exceed 100 ft. The measured flow of this stream during the past five years gives a maximum discharge of 9 620 cu. ft. per sec. in April and a minimum of 35 cu. ft. per sec. in September, but both figures may be considered to be abnormal, the average extreme flood discharge running between 2 000 and 3 000 cu. ft. per sec., with low water from 50 to 100 cu. ft. per sec. It was fully realized, however, that the new bridge and approaches must be designed to withstand the most extreme conditions recorded in the past or possible in the future.

The geological formation of the bed of the Capilano River presents serious difficulties for obtaining good foundations, consisting of gravel and large round boulders of undoubtedly glacial origin, for an indefinite depth. On this the hydraulic force of the river exerts a "digging" action which was given practical

* Victoria, B. C., Canada.

demonstration during 1917 when the east span of the Marine Drive Highway Bridge collapsed, owing to the undermining action of the river in the abutment foundation. This is a highway bridge of reinforced concrete, consisting of four 70-ft. arches, and was built by the municipality in 1914; it is less than a mile up stream from the Pacific Great Eastern Railway Bridge and is referred to here by the writer because river conditions at both bridge sites are exactly similar, identical methods having been utilized for protection work.

The contract was let in April, 1918, for a bridge consisting of two 100-ft. through Howe truss spans supported on a center pile pier, with a pile abutment at either end, both substructure and superstructure being of the standard type usual for this class of railway bridge, and pile-driving was started during same month. Some delay was encountered in the erection of the superstructure, owing to the falsework being carried out by a sudden freshet, but the bridge was opened for traffic on November 7th, 1918.

Although the writer has not been able to secure much reliable data as to penetration, it is significant that nearly all the piling pulled out in connection with the removal of the original trestle, was found to have "broomed" at the points. The average "refusal" depth of the piles driven on the new work was about 6 ft., with a maximum of 10 ft., but in such a river-bed formation it is doubtful whether any pile was driven beyond 6 or 8 ft. without "brooming", although shod and ringed.

When construction was started in April, the main channel of the Capilano River hugged the west bank which was found to be rapidly eroding and, consequently, endangering the fill at the west end of the bridge toward which the flow of the river was directly working. To remedy this condition three log wing dams (or groins) were built about 400 ft. apart, up stream from the bridge, and approximately at right angles to the current; the dimensions were about 18 ft. long by 6 ft. high and 4 ft. wide, each crib resting on a brush wire mattress. The principle involved being flexibility as opposed to rigidity, may be explained as follows: A brush mattress is first made from ordinary No. 9 wire fencing (such as is used for right-of-way fences), with live green tree limbs, willows, or any suitable brush in leaf, laced in and out between the strands; this is sunk to the bed of the channel as a foundation for the cribwork which is built up similarly to a log shack, except that the timbers are wired together with No. 8 or No. 9 telephone wire, and drift-bolts are driven where required.

Before filling the cribs, the wire fencing is dropped inside to form a loop with the ends projecting above the sides, and loose brush is laid in the bottom of the crib (on the wire fencing), this being covered with gravel and boulders up to the level of the crib top. The shore end of the crib sides should be dug into the bank for 2 or 3 ft. and protected with a wire and brush mattress at the up-stream side, from top to bottom. The height of the crib is governed by high-water flow which should have a depth of from 18 in. to 2 ft. above the top, to avoid impact from floating logs and other detritus.

The action of the wing dams is to cause a strong eddy which carries in gravel and boulders behind the cribwork, forming a bar, and in the writer's experience this constitutes the best type of protection, as a river seldom digs out bars of its own, under normal conditions. As a general rule, the up-stream toe of the crib

undermines more or less during high water, causing it to cant over at any angle, and the cribs built on the Capilano River proved to be no exception, but, being flexible, adapted themselves to the new position while the brush and wire beneath the filling followed the subsidence down, checking further undermining; one wing dam crib, exposed to the full force of the current at a bend in the river, was undermined at the up-stream face to such an extent as to tilt forward 60° from the vertical, but without developing any lateral movement. In this case, a new crib was built up behind the original structure which now acts as a foundation protection, effectually preventing erosion beneath the toe. Within a few months of construction the wing dam cribs had built up a solid bar along the west bank of the river which then changed its course to the opposite bank, threatening the fill at the east end of the bridge and emptying the east abutment by scouring beneath the sheeting. Prompt action being necessary to meet this contingency, continuous loops of wire fencing were dropped inside the empty abutment, followed by a heavy layer of green brush on which loose rock (train-hauled) was dumped to complete the filling; in addition to this, a brush wire mattress was woven and sunk vertically down the nose of the abutment, the ends being swung into place along each side by the force of the current.

Similar methods were adopted at the center pier, which was undermined and emptied shortly afterward, the river also scouring out a hole 16 ft. deep immediately behind it, which was filled as follows: A wire brush mattress was sunk for a foundation and was followed by heavy loose rock filling 5 ft. in height, wire fencing laid on top completely covering the surface of the filling; the second layer of loose rock and boulders for the next 5 ft. was also covered with wire fencing, and the final 6-ft. section, bringing the fill level with the river bed, was mattressed about 18 in. below it.

In addition to this work on the piers, the east bank was mattressed for a distance of 900 ft. up stream, as the current was commencing to find a new channel through the river flats which lie at an elevation of from 8 to 10 ft. above low-water level; all the work, as described, has been subjected to the most severe flood tests during the past 1½ years without showing any sign of erosion. The use of plain wire fencing in connection with gravel and boulder filling for wash-outs, cribs, etc., has proved remarkably effective both in checking erosion and providing a stable foundation base for the extension of the protection work when required, as the fine silt deposited during high water rapidly fills in all the voids, forming a solid mass which is impervious to the destructive "digging" action of the river. Similar wing dams composed of gravel and brush only, owing to the scarcity of timber, have been built on prairie streams under the writer's supervision with satisfactory results, but in every instance the channel bed formation consisted of gravel and boulders in which tests made during flood discharge established the fact that a down-stream movement was perceptible in the gravel for a depth of several feet below its surface. It is this feature which constitutes the greatest menace to bridge foundations, as any rigid structure (such as a pier) forms an obstruction providing the necessary leverage to set up gyratory action wherewith the hydraulic force of the river bores downward, finally undermining concrete substructures or digging out the piles in timber construction. In theory, the engineer is supposed to lay out his work so that excavation will be carried down until a stable foundation is assured, but this discussion is designed to show that

alternative methods are both feasible and practical, especially when the amount of expenditure is limited.

In conclusion, it may be stated that while no hard and fast rules can be laid down for this class of river-protection work, there are a few essential points which should not be neglected, namely:

Wing dams should not be too long, as this prevents the "eddy" from carrying material inshore to the bank; a length of crib of from 16 to 20 ft. will start a good bar, and when this is built up the wing dam can be lengthened in sections to continue the same operation. The height should be carefully regulated to allow for a depth of from 1½ to 2 ft. of water flowing above the top logs at flood stages of the stream.

Flexibility being the underlying principle in this type of protection work, the logs should be held together, both vertically and horizontally, with several strands of twisted No. 8, galvanized wire; drifts should be used as little as possible, owing to their tendency to split the logs when the crib becomes distorted from undermining.

An inside width of about 4 ft. is sufficient for the first section as extensions are added by the logs overlapping on the outside, thus widening the crib as each new section is built on.

The selection of strategic points at which to place the wing dam cribs is a matter of judgment based on a careful study of conditions, and as the construction has to be carried out at low-water stage the engineer should be ready personally to observe the action of the stream during the first high water, in addition to the after effect when it has lowered sufficiently to expose the cribs, etc. It will then become evident whether these are properly located or working to advantage, and changes or additions can be made in accordance with the results revealed.

This class of work closely resembles a game of chess in which the river is constantly making new moves and calling "check", while the engineer responds with countermoves backed by the definite object of being able to say "checkmate."

J. A. Ockerson,* Past-President, Am. Soc. C. E. (by letter).—Mr. Curd's bank protection work near Natchez, Miss., is the first of its kind in that form along the Mississippi River; mud-cells have been in use, however, for the collection of silt on other streams. Deposits can be induced in sediment-bearing streams by any device which will diminish the velocity of the current. On the Missouri River a device called a "retard" has recently been put into use, which is said to be effective in arresting and preventing bank erosion. This consists of trees anchored to submerged piling, forming a permeable dike extending out into the river normal to the bank.

The writer examined Mr. Curd's work during the low water of 1920 and found it to be effective. Although mud cells can be relied on to gather sediment, there is no permanent stability in the cells themselves, and it is believed that the life of such construction is necessarily brief; the willow baskets will decay, and the mud gathered will be again taken up by the current and carried away.

In the case described by Mr. Curd, the accumulated sediment is covered with concrete, which will add much to its stability. There is also a pile protection dike just above the work, which is really the key to the situation. Should that fail, it is probable that the mud cells would go with it.

* St Louis, Mo.

The site of the work is especially favorable, since the construction rests on a bed of conglomerate which resists erosion, and under-cutting or sloughing of the base is impossible. There is no other case like it on the Mississippi River between the Ohio River and the Gulf of Mexico and the engineer promptly took advantage of it. The work as a whole, the results obtained, and the very low cost thereof, are all highly commendable.

Mr. Macintyre says "a river seldom digs out bars of its own" make. This is not true of the Mississippi. This river has a habit of building up bars that in the course of years are covered with timber only to be carried away again during a single flood period.

The kind of bank protection to be used depends on the character of the stream. A sedimentary stream flowing in a bed of its own formation requires radically different treatment from one of any other character, and the same is true of shallow and deep streams. The Mississippi River opposite Mr. Curd's work is about 80 ft. deep at low water. In such depths cribwork is impracticable, both on account of excessive cost and the physical difficulty of placing it.

On the Mississippi below the Ohio River, the only effective means of holding a caving bank is to cover it with a substantial willow or concrete mat from the low-water line out to, and well beyond, the thalweg, grade the upper bank to a slope of 1:3, and pave this slope with concrete or rip-rap laid by hand.

WILLIAM G. ATWOOD,* M. AM. SOC. C. E. (by letter).—The method of protection, so ably described by Mr. Curd, is one of the many attempts being made, as indicated by Mr. Macintyre, to use the destructive forces of a river for beneficent purposes. Another method along this same line which has been developed in the Missouri River by the Woods Brothers Construction Company, takes advantage of the same factors. The experience of river engineers has shown that the most efficient method of protection is one which guides rather than directly opposes the course of the river and which, consequently, results in a diversion of the current and the formation of a bar in front of the caving bank. The system of permeable dikes developed by the Army engineers and others will secure this result as long as the dikes are not destroyed. These dikes are not flexible and are subject to destruction by erosion, ice, and heavy drift.

The system of "retards" developed in the Missouri River depends on this same principle of sedimentation by slowing up the current, but it is not injured by the destructive forces previously mentioned. The system is based on the use of the Bignell concrete pile as an anchor. Owing to the features of this pile, it can be sunk below the bed of the river, thus forming an anchor, as stated.

Before the pile is sunk, from ten to twenty ¾-in. cables are attached to its head, the free ends of these cables being carried ashore or on to a barge, trees from 8 to 24 in. in diameter are made into rafts containing fifteen or twenty trees by taking a half hitch around each tree and stapling it. Each of these rafts is attached to one of these cables on the pile, and as many of them are used as is necessary to fill the river from its bed to the surface of the water.

These retards are located in series from 300 to 1 000 ft. apart, and extend into the river from the bank from 100 to 400 or 500 ft. Their use has resulted in the formation of bars almost immediately after they have been placed, and in every

* St. Louis, Mo.

case where they have been used in the Missouri River during the past three years, they have stopped bank erosion.

The cost of this system, where suitable timber is available, is much less than the cost of mats, and there is every indication that it is more effective. The system has also been used for the protection of mats which were being destroyed by the current. A notable example may be seen on the Iowa bank of the Missouri River just above the Plattsmouth Bridge of the Chicago, Burlington and Quincy Railway.

Limitations to the use of this system will occur, of course, to river engineers, but it is believed to represent a distinct advance in the treatment of the problem and one which will have very general application.

W. C. CURD,* M. AM. SOC. C. E. (by letter).—The writer feels obligated to the discussors for their friendly and interesting consideration of his paper. It is gratifying to observe that a bond of fellowship may sometimes exist between those who have dealt with difficult problems along somewhat similar lines.

The Natchez problem required the writer's attention during its entire development, and the final outcome demonstrates how trivial, after all, may be the obstacles which arise from time to time on any work. No credit is claimed for originality in design, but only for the application of established principles to the successful elimination of an annoying situation.

In closing the discussion it seems desirable to answer some of the questions raised concerning the application and life of mud-cell construction.

The purpose for which the mud-cell dike was installed at Natchez was to collect sediment and hold it in place until it could be protected with concrete paving. Ample deposits of silt were secured before the life of the cells was reached, and therefore nothing could have been gained by providing facilities for repairing or extending such life. A double row of piles at the toe of the cell wall, as suggested by Mr. Brumley, would not have added security to the work, owing to the peculiar and non-erosive character of the foundation, but, on the other hand, a large increase in cost would have resulted.

The short life of mud cells is referred to by Mr. Ockerson. The life of any brush construction placed above low water is dependent on the diameter of brush used, but such life, in any event, is short. Where mud cells are submerged at all stages they can be made as durable and as stable as other forms of brush construction. Ample proof of this statement may be found at many places.

Although the Natchez problem was greatly simplified by natural and ideal foundation conditions, the same principles could have been applied to an erodible bed by strengthening and widening the foundation mattress on which the cells were built.

The writer's experience with protective work on alluvial streams prevents him from being fully in accord with the views expressed by Messrs. Macintyre and Atwood concerning the value of dikes for permanent protection. Numerous types of dikes have been developed in past years, many of them of considerable merit, but the usual limitations in application have relegated most of them to the discard. Dike work on rivers is now confined principally to channel contraction, and the question of permanent bank protection is left to the continuous mattress, of which no better examples can be found than along the Mississippi River.

* Chicago, Ill.

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Paper No. 1469

SOME INVESTIGATIONS AND STUDIES IN HYDRAULIC-FILL DAM CONSTRUCTION

By J. Albert Holmes,* M. Am. Soc. C. E.

With Discussion by Allen Hazen, M. Am. Soc. C. E.

Synopsis.

This paper contains the results of a number of studies and experiments made in connection with the building of earth dams, and describes the construction methods used and results obtained. It includes a discussion of the importance of careful examination of foundation conditions, the determination of the character of materials to be used, the methods of construction suitable to the materials, and the object and utility of the investigations and the manner of making them.

Details are given of the Somerset Dam, Paddy Creek Dam, and others of the Bridgewater, N. C., development, with the hope that the information will be of assistance in future design and construction of earth dams and lead to still more careful investigations.

The writer recognizes that the compression and seepage tests described could well have been carried to greater pressures in a laboratory with suitable apparatus, and more extensive results obtained. The apparatus used was limited in the amount of pressure that could be applied. It is also suggested that another desirable line of investigation is to determine why, and how, small percentages of vegetable or organic matter affect porosity so notably.

In recent years a number of high earth dams with fine earth cores, placed by the semi-hydraulic method, have been built in the Eastern States and others are in process of construction.

In the West, topographical relief has permitted both the transportation and placing of material in earth dams by hydraulicking; in the East, differences in elevation are seldom sufficient to afford the necessary sluicing grades. These conditions have been met by excavating from borrow-pits near the level at which the material is to be delivered in the dams, and transporting such material by dump cars to trestles from which it is dumped and sluiced, or to sumps at the

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* Somerville, Mass.

base of the dam, where it is mixed with water and pumped into place, each project, of course, being a problem in itself.

Reference has been made recently in the *Transactions** of this Society to the Somerset and Paddy Creek Dams.

The writer was connected, as Resident Engineer, with the building of the Somerset Dam for the New England Power Company and as Resident Engineer for the J. G. White Engineering Corporation, on the group of dams of which the Paddy Creek Dam was one, for the Southern Power Company. The J. G. White Engineering Corporation also acted as Consulting Engineers for the Somerset work.

The Somerset Dam, Figs. 1, 3, and 6, built in 1912-13, at Somerset, Vt., is in the region of glacial drift. The material is a boulder clay, and the method of placing was to excavate by steam shovel, transport in cars, dump from trestles (Fig. 2), and sluice into a central pool in which the core is formed. The borrow-pits were at or above the elevation at which material was delivered in the dam. Sluicing from the dikes (Fig. 3) formed at the trestles left boulders and other coarse material in the dikes, making them pervious and stable. By this method the only limit to the size of rock placed in the dikes is the capacity of the steam shovel to load them.

The dam to be built by the City of Providence, R. I., is also in the glacial region. Very careful studies have been made of the materials and methods of construction to be used. The glacial drift of the region is thin and gravelly, and suitable material is not abundant. The core will be formed of top-soil and subsoil and, because of the expense of obtaining this soil, it will not be sluiced into place, but will be put directly in position, dampened, and rolled as in the case of the core of the North Dike at the Wachusett Reservoir. The core will be comparatively thin and will be supported by gravel dikes. Conditions have dictated the method. This is not even a semi-hydraulic-fill dam. It is mentioned here because the investigations proved that this dam should not be built in that way and because of its thin core.

Beyond the southerly advance of the ice sheets, where erosion, separation, and redeposition by melting ice fields have not taken place and where no solid rock has been ground into rock flour, still another problem is encountered. In the group of three earth dams recently completed for the Southern Power Company at Bridgewater, N. C., Paddy Creek (Fig. 4), Catawba River (Fig. 11), and Linville River, one of the problems was to get into the cores of the dams a sufficient quantity of fine material and, at the same time, not carry in the coarser parts. These dams are in a non-glacial country. Here, the rock ledge is found in its original position, decayed and softened to considerable depths by the dissolving and leaching away by water of much of its structure. Although residual clay is found in the depressions between the lesser ridges, deposited there from the ridges themselves, this material constitutes a very small proportion where the movement of large quantities of earth is involved. Moreover, it is far from being a pure clay, in that it contains much quartz sand and other less hard particles. Although there is little extremely fine, neither is there coarse, hard rock in material of this

* "Hydraulic-Fill Dams," by Allen Hazen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 1734, 1809.

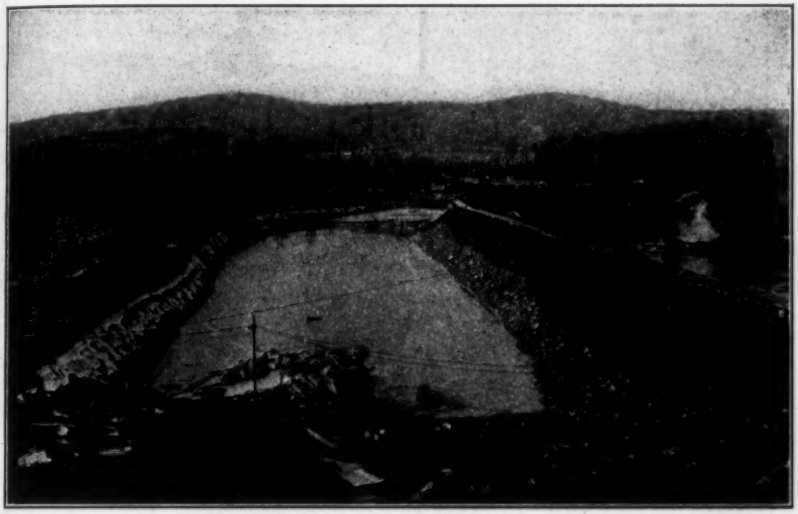

FIG. 1.—SOMERSET DAM: VIEW FROM EAST END, PUMPING STATION AT RIGHT TAKING SLUICING WATER FROM RESERVOIR.

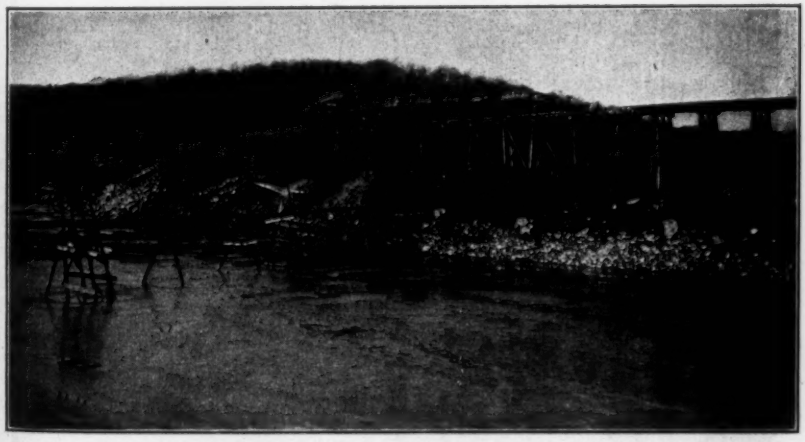

FIG. 2.—SOMERSET DAM: DUMPING FROM TRESTLE AND SLUICING. SLUICING MAIN CROSSING POOL IN FOREGROUND.

FIG. 3.—SOMERSET DAM: SLUICING MATERIAL FROM DIKE TO CORE. NOTE SIZE OF ROCK REMAINING IN DIKE.

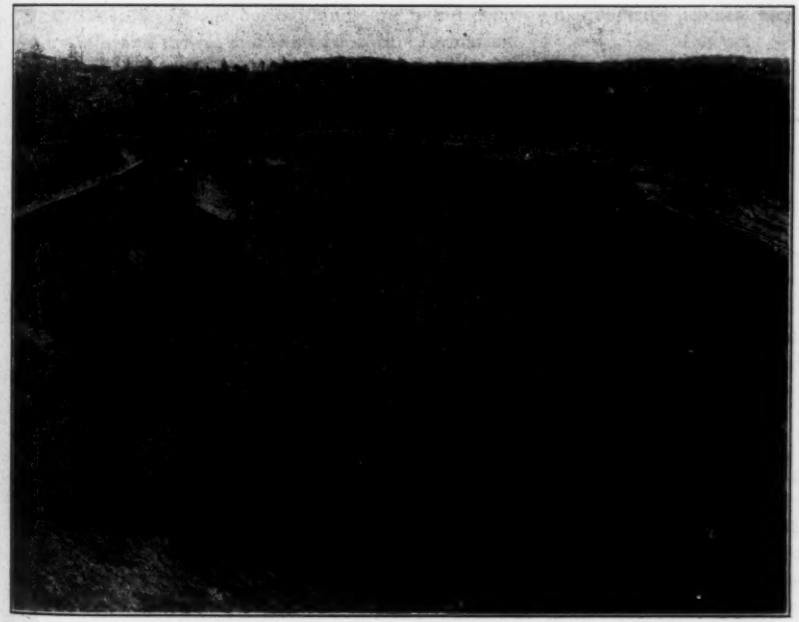

FIG. 4.—PADDY CREEK DAM: POOL AND DIKES 7 AND 8, LOOKING EAST.

character; excavations in the harder portions afford no fine material and, therefore, cannot be used.

Geologically, the formation of the region in which these dams are built is very old, the greater part being of the Archean period. The ancient rock of this period is a gneiss. It is highly metamorphosed, and in the process has lost all traces of its original character. On this formation, both solid and decomposed, the dams are built. The dip of the strata is between 20° and 30° southeast—down stream—and the strike is nearly parallel with the dams. Water readily finds its way into the numerous joints, and decomposition takes place along the ridges to depths of 80 to 100 ft.

A remarkable thing about this decayed rock is the loss in weight it has sustained in the process of decay. The underlying solid ledge weighs about 170 lb. per cu. ft., while the thoroughly decayed rock weighs only 72 lb. per cu. ft. as it is found in place in the borrow-pits.

The neighboring high mountain region, in the eastern foot-hills of which the dams are built, lay for long periods of time below the level of the shallow continental seas. Beginning in early Cambrian and extending through Silurian and, perhaps, to a later time, sediments were deposited in this sea. Among other formations that remain after elevation and partial erosion of this sedimentary strata is a quartzite of the lower Cambrian Age.

In the beds of the creeks and streams issuing from the mountains and underlying the silts of their bottom-lands are found deposits of quartzite pebbles, large and small, and gravel and sand from this formation.

At the dam sites the quartzite pebbles were also found on the hillsides above the present river beds, deposited there by the water as it cut its channel through the rock, such deposit always being found directly on the decayed ledge. At each of the dams the river channel is on solid rock, and in each case the river has cut through the ridge at the junction of a hard and softer formation and, in shifting across the valley bottom, has deposited a layer of gravel and boulders from 2 to 5 ft. thick, buried under from 5 to 20 ft. of silt. The gravel layer is level across the valleys and at the same elevation as that in the existing channel. It is highly water-bearing, and had to be removed under a portion of each dam.

While the rock was sound and hard in the existing river channel, it was not so under the gravel deposits of the bottoms, although it lay at the same elevation. Here the bed-rock was completely decayed and disintegrated, and the same was true of that under the gravel on the hillsides.

Investigations at Dam Sites.

Careful study has characterized the investigations preliminary to planning and building each of the dams mentioned. The dam sites and borrow-pits were explored by means of test pits and tunnels and by wash, pump, auger, churn, shot, and diamond drill borings. Of these, the shot and diamond drill borings in rock, and the pits and tunnels and the churn drill borings in earth, are the most satisfactory, and the wash borings the least. In fact, the writer considers wash borings so unsatisfactory in results as to render them unfit for use, and the churn drill is reliable only when samples are taken with a drive pipe.

At Somerset and on the Bridgewater dams, permeability of the earth on the dam sites was investigated by filling test pits with water and noting the rate of seepage. The pits were measured at several elevations, the area of the sides was determined, and the results were plotted as a curve for each pit. Water for filling a pit was pumped into a stilling-box from which it passed over a measuring weir and down a wooden box conduit to prevent washing the sides of the pit. Frequent weir readings and elevations of the water surface in the pit were made during the filling, and the water surface measurements were continued after pumping had stopped. These data were plotted in the form of curves, Fig. 5, and in their proper relation as to elevation with the pit and the pit area curve. The gallons pumped were determined from weir measurements and the gallons in the pit from elevations and pit volume. The seepage curves were plotted on a time-volume basis of gallons per minute.

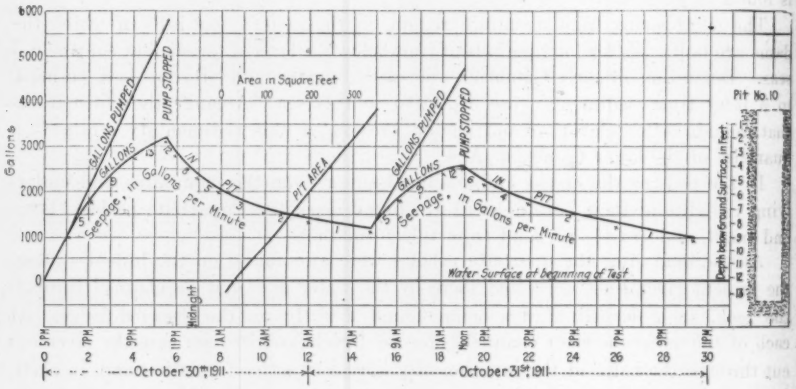

FIG. 5.

To illustrate, using the period between 7 P. M. and 9 P. M., October 30th, it will be found from the curve that 2 100 gal. were pumped, and that there was an increase of 1 000 gal. in the pit, making a loss by seepage of 1 100 gal. during the period of 120 min., or at the rate of 9 gal. per min. The rise of water in the pit during the period was somewhat more than 3 ft. The area curve gives 230 sq. ft. below mid-height of this rise, and 0.04 gal. per sq. ft. per min. as the average seepage through the material in the sides of the pit below this elevation.

The water as it rises in the pit has first to fill the voids in the surrounding material. Such filling of voids continues as long as the rise continues, and, for this reason, the rising curve does not indicate the seepage rate as well as the falling part of the curve. The rates for the latter were determined by taking from the curve the loss in gallons in any time interval and dividing by the minutes in that interval, and then by the pit area, in square feet, below that elevation as given by the area curve. This gives gallons per square foot per minute, as before, and is also the average for the material below that elevation. The form of the seepage curve is due both to increased head and greater porosity as the surface of the ground is approached.

The average rate at which water is passing out of the pit is the rate at which it is passing through the surrounding material and through each foot of average material. In order to maintain the flow through 1 cu. ft. of material in the side of the pit, the same quantity of water that passes into that particular cubic foot from the pit, must pass into the adjacent foot of material. The rate of seepage per square foot per minute from the pit is the seepage per foot per minute through an average square foot of the surrounding material, and this rate may be applied in direct proportion to greater heads and inversely to greater thickness of material.

The test pit under discussion was located in a gravelly knoll close to the center line of the dam and within the area of the core, its top being 33 ft. below full reservoir, and it was desired to know the amount of seepage that would take place through the material of this knoll under the head due to full reservoir. As a result of this investigation, the knoll was trenched to a point below the bottom of the pit and the excavation, being part of the cut-off trench, was filled later with core material as the latter was sluiced into place.

On the Bridgewater dams, the pits on each dam site were tested for seepage from the pits, and the depth of the center-line trench was determined from the results. At one of the pits, excavated in a hill between the south and main Catawba River Dams, the filling and seeping-away process was repeated four times.

For measuring the flow from stilling-box to pit, triangular and 6 and 12-in. Cippoletti weirs were used. In order to obtain the best results from an investigation of seepage from a test pit, it is necessary that the pit be regular in outline and, if it is reduced in cross-section as excavated, that the reduction be uniform. A tight board floor should be laid in the bottom of the pit to eliminate seepage there and confine it entirely to the walls of the pit.

It would be desirable to refer all elevations in the pit, and of water surface, to the datum established for the dam, and in measuring the pit for surface area and volume, to take measurements at regular intervals above the datum. The water supply should be sufficient to fill the pit quickly.

The most valuable data are those obtained after the pit is filled, or, as previously mentioned, those taken from the falling portion of the seepage curve and, therefore, this part of the experiment should be most carefully made. Adjusting the flow into the pit so that it just balances the seepage at various elevations or at critical points where the character of the ground indicates a change in seepage rate, is another method of obtaining the information given by the curve, but this method consumes much more time.

The examination of the material underlying the dam site at Somerset was made both by test pits and churn drill. The drift is unmodified or unstratified and contains boulders, gravels, sand, and clay. The sand and gravel occur in pockets or lenses of no great extent, and the main body of the drift is boulder clay down to bed-rock, 40 ft. below the river channel. Four of the thirty-eight pits on the dam site were carried to a considerable depth, a maximum of 44 ft., and, later, five others were supplemented by drilling to and into bed-rock, using a well drill—churn drill—and 6-in. casing and taking dry samples with a ¾-in. drive pipe. All borings were investigated for seepage into and from the ground, each layer of sand and gravel being tested separately. When a porous stratum was encountered, the method was to raise the casing slightly so as not to interfere with the flow

and note the elevation to which the water rose after a considerable period. The water was then bailed from the casing, measured, and the rate of flow from the sand determined. Following this, the water was returned to the casing in order to find out at what rate the sand would take water after its storage supply had been removed. The rate of flow from the sand-pockets ranged between 25 and 95 gal. per hour, and the quantity that could be returned to the sand was from 4 to 13 gal. per hour. In all cases the water-bearing strata were overlaid by a considerable depth of clay, and the accumulated water stood always higher than the adjacent pond and pool surfaces, showing that there was no connection therewith to produce the head.

The Somerset Dam.

The building of the Somerset Dam, Fig. 6, has been described in considerable detail elsewhere,* and, therefore, only the methods used are mentioned here. In placing the core, the object was to deposit fine material at the base of the dam over at least one-third of its width, reducing this width considerably up to 30 ft. above the base and less rapidly for the remainder of the height. There was no question about the core material being sufficiently fine. Examination of the borrow-pits indicated the quantity of core material available, but it was desirable to know just where in the dam the core was being deposited and its extent, whether or not the proper width was being maintained, and to make sure that layers of sand and gravel were not cutting through the core from side to side. In order to determine the character of the material below the surface of the pool, samples were taken on cross-sections 300 ft. apart, the location of these samples on the maximum section of the dam being shown on Fig. 6, as well as the outline of the pool and core. The monthly progress is shown by the numbered areas on the section.

After sluicing operations had been carried on for a time, an estimate was made to determine the proportion of fine material being sluiced from the dikes to the center of the dam. The volume investigated amounted to 85 000 cu. yd. and gave a ratio of 1 : 4, or 20% of fine material. The approximate line separating the coarse from the fine material was assumed to pass through the point where the steep sides of the pool changed to flatter slopes, and this assumption was supported by samples taken from the pool bottom. A sluicing experiment was also made in a tank to determine the quantity of fine material it was possible to remove from a sample taken directly from the borrow-pits, and the proportion of very fine material removed was found to be 20 per cent. This did not constitute all the fine material, but it did show how much the sluicing would remove and meant that one-fifth the cross-section of the dam would be fine material, and constitute the core.

The one-fifth or 20% line was projected to the top of the dam, and is shown on Fig. 6 as a dotted line. The section shows how near the core came in location and area to that proposed, and that the methods used were the correct ones to bring this about and, at the same time, by removing from the outer dikes the greater part of the fine material, to make them porous and stable.

Care in keeping the sluicing faces of the dikes parallel and at equal distances

* "Preliminary Investigations, Construction, Sluicing, and Core Sampling at the Somerset Dam", by J. Albert Holmes, *Engineering News*, Vol. 71 (1914), p. 1236.

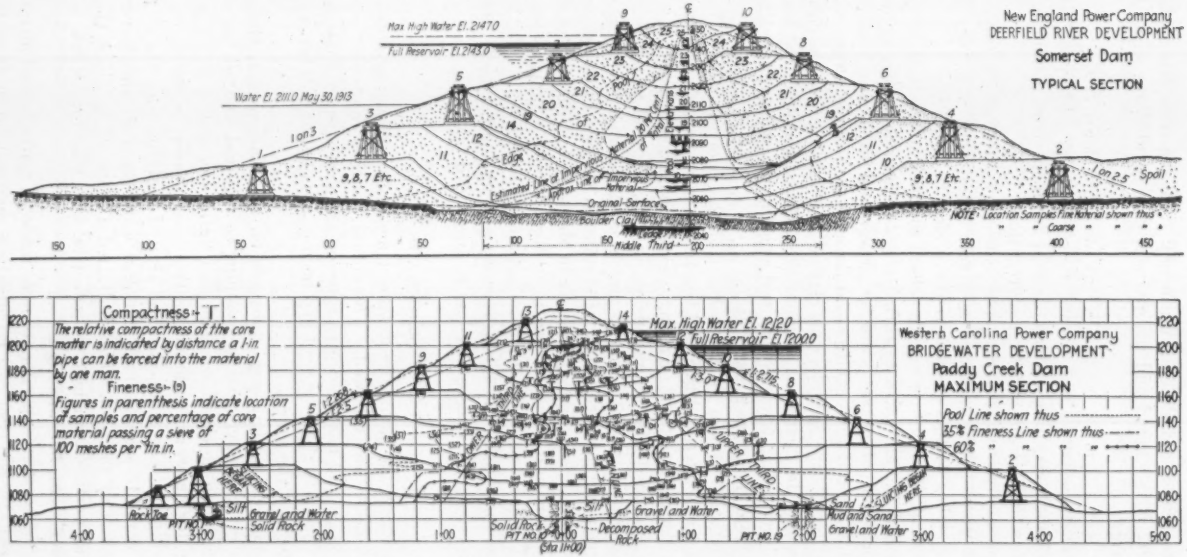

FIG. 6.—CROSS-SECTIONS OF SOMERSET AND PADDY CREEK DAMS.

from the core center and sluicing uniformly from opposite dikes, places the fine material where it is wanted and maintains a core of uniform width from end to end of the dam.

Sampling shows that the coarser grains were laid down at the edge of the pool and diminish somewhat in size toward the center. A wide pool causes greater segregation, though very fine material is laid down in short distances. The depth of the pool has little or no effect on segregation.

Mechanical analysis of the Somerset core gave 94% of the material passing the 200-mesh sieve. The material was free from sand and very smooth, and advantage was taken of this characteristic to classify it quickly.

The method of determining rapidly which samples, taken from the pool bottom, should be classed as "fine" was to thrust a pencil into the material in the sample bottle, the presence of coarse sand or gravel and even fine sand being easily detected in this way; if the pencil passed through the sample several times without encountering "grit", the sample was classed as "fine". From a combination of these samples and the change of slope of the sides of the pool, the location of the dot and dash line on Fig. 6 was determined as being the approximate line separating the fine core from the coarser material.

Careful elevations taken on the bottom of the pool after work had been discontinued for the winter and repeated before the next season's work began, showed no change, either of settlement or rising, though the dikes themselves settled. Very fine material remained long in suspension, in fact, the pool water never was clear, and it was estimated that at times as much as 100 cu. yd. per 24 hours was lost through the pool overflow. Sulphate of aluminum was used to precipitate this fine material, partly by suspending it in bags in the pool, but mainly by the introduction of a solution into the force main at the sluicing pump house. Continued sampling at the overflow indicated a reduction of about 50% of the loss. During the second season's work, the dikes were carried up more rapidly, and there was little or no overflow and consequent loss of suspended material.

The Bridgewater Dams.

On the Bridgewater dams, the Somerset method of dumping from trestles and sluicing from dikes was followed. The actual sluicing, however, was done from scows in the pools and not from pipe lines on the dikes, as at Somerset, except in finishing the final lift after the pool had been eliminated or had become too narrow and shallow to float the scows.

Seepage and evaporation from the pools on the Bridgewater dams, amounting to about 0.4 sec-ft., was replaced by pumping as required, and all fine material was retained, there being no overflow. The pool water was seldom roily, in fact, it was notably clear during warm summer weather and clearer in the pools than in the river from which it was pumped. The sluiced material and the slight roiliness caused by it, could be seen through the clear water as it advanced over the bottom toward the center of the pool. This indicated a material in which there were few extremely fine particles, though its effective size was low, 0.026 mm., and its uniformity coefficient, 5.6, was high.

A number of seepage experiments were made in tanks in the manner described by Allen Hazen, M. Am. Soc. C. E., utilizing material taken from the dams and

borrow-pits. The apparatus used was not satisfactory, the results giving relative values as between different materials, but not absolute seepage quantities. The experiments showed where a certain material might best be used in the dams, whether in the porous outer dikes or for cores.

As previously stated, one of the problems on the Bridgewater dams was to get into the cores a sufficient quantity of fine material. To make certain that this was being done, numerous and frequent samples were taken from the bottoms of the pools. These samples, about ¼ lb. in weight, were obtained with a cone-shaped tin can attached to the side, and close to the end, of a sounding pole. The samples were taken on cross-sections not more than 300 ft. apart, and their location in the section and elevation was determined.

The Paddy Creek Dam.

Fig. 6 shows the maximum section of Paddy Creek Dam, and is typical of all the dams in the group. The location and fineness of the samples taken on this section are indicated, the location by the parentheses and the fineness by the figures within the parentheses, the figures being the percentages passing a sieve of 100 meshes per lin. in. Other data on the section are the lines showing the extent of the pool, the position of material finer than 35% and 60% passing the 100-mesh sieve, and the location of the trestles from which the material was dumped.

More than 2 000 samples were taken during the progress of the work. Mechanical analysis of some of these samples was made, but complete analysis of all the samples consumed too much time—the information they furnished must be made use of at the earliest possible moment and to do this a more simple method was adopted. This was to determine their value by comparison with materials of known fineness and impermeability.

The mechanical analysis curves, 4, 6, 9, and 10, Fig. 7, represent materials that have been experimented with and found to be satisfactory core material. The curve, 1 to 8, is a combination of the analysis of samples of six different borrow-pit materials to be used in the dam.

It is desired to have a core equal to one-fifth or 20% of the volume (or cross-sectional area) of the dam. The curve, 1 to 8, represents the material with which the work is to be done or the dam built, one-fifth of which will be core. The finer fifth of this material is represented by that part of the curve lying below the 20% by weight line on the diagram, the larger particles of which have a mean diameter of 0.13 mm. A line projected vertically from the point where the combination curve crosses the 20% by weight—also volume—line, divides the curves, 6, 9, 4, and 10, at their 0.13-mm. diameter point. Therefore, in order to show that a core as good as would be obtained by using materials, 6, 9, 4, and 10, is being obtained from the 1 to 8 material, samples may be taken from the core and passed through a single sieve, in this case a 140-mesh sieve, which passes material having a mean diameter of 0.13 mm. The percentages that must pass this single sieve may be taken from the diagram. The points where the vertical one-fifth volume line intersect the curves have the following values on the percentage by weight scale: Curve No. 6, 30%; No. 9, 35%; No. 4, 39.5%; and No. 10, 70%. This means that to obtain a core from the combination material equally as fine as

that from the No. 6 material, 30% of the core from the combination material must pass the 140-mesh sieve, and so on.

As described previously, samples were taken from the bottom of the pool, dried, and passed through one sieve, and the percentage of material passing the sieve was noted. On the Bridgewater dams a sieve having 100 meshes per lin. in. was used, and the minimum for core material was set at 35% passing this sieve.

In screening core materials it is not possible to follow the methods used in the examination of sands and gravels and, ordinarily, only those sizes passing the No. 10 sieve (2 mm. in diameter) are considered. A large percentage of this material passes the 200 sieve (0.10 mm. in diameter), and the finer sizes are determined by

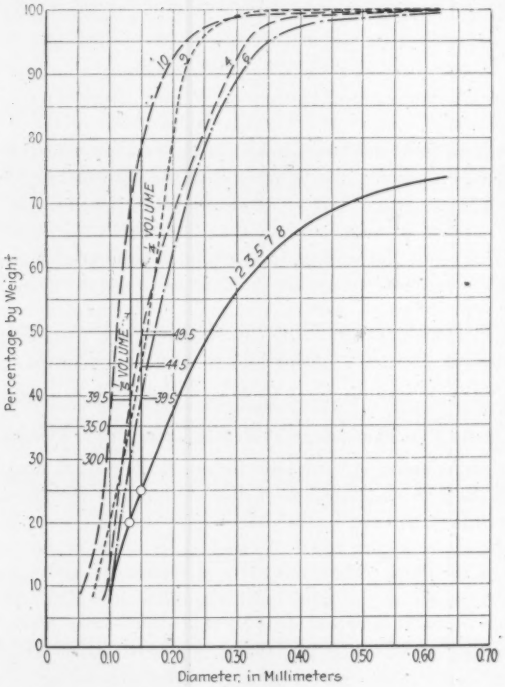

FIG. 7.

elutriation. These finer sizes of a sample adhere to the wires of the finer sieves, clog the openings, and cannot be gotten through during a fixed period or by a certain number of "shakes".

The sample must be removed from each sieve of the "nest" on which it has clogged, and the sieve cleaned with a brush or by rapping, the cleaning process being repeated until no clogging occurs and the wires of the sieve show clean and bright after the final shaking. Lumps formed in drying that do not break down in the sieves, may be reduced under slight pressure applied with a cork. Lumps that resist this pressure will readily break down in water.

The extent of the 35% material is indicated on the section, Fig. 6, and is considered to be the core of the dam. This area is referred to as "35% fine" and a still smaller area as "60% fine". All samples containing 35% and more passing the 100-mesh sieve were reserved for further analysis and experiment.

Mechanical analysis was made of the combined core samples of the three large dams. The effective size and uniformity coefficient, together with the area of pool and core, are given for each dam in Table 1.

TABLE 1.

Dam.	Core.		Areas in Percentages of Maximum Cross-Sections.		
	Effective size.	Uniformity coefficient.	Pool.	35% fine.	60% fine.
Catawba	0.026	5.6	42	42	24
Paddy Creek	0.030	5.2	48	31	21
Linville	0.028	5.4	52	27	20

In the Catawba Dam there was no material inside the pool lines finer than 35%, so the area for this size appears in Table 1 the same as for the pool, namely, 42 per cent. It will be seen from Table 1 that the "60% fine" areas came nearer the one-fifth or 20% cores sought than the "35% fine".

In sluicing the core into place, there is always the danger of carrying a layer of sand or coarser material partly or entirely across the pool and connecting with other coarse material. Even though one-fifth the area was sufficient, and "35% fine" was adopted as the minimum, material of this size was watched very closely when it began to pass within the limits of the middle third.

An additional mechanical analysis was made of a mixed sample, composed of 266 core samples from Paddy Creek Dam. All the 266 samples were those containing more than 35% passing the 100-mesh sieve and were taken on several sections and throughout the full height of the dam.

The smaller sizes were determined by water elutriation. The analysis was made by Mr. H. W. Clark, as follows:

Percentage by weight finer than:

6	mesh	sieve	or	3.89	mm	100.0
10	"	"	"	2.04	"	99.4
20	"	"	"	0.93	"	93.7
40	"	"	"	0.46	"	89.0
60	"	"	"	0.316	"	81.2
100	"	"	"	0.182	"	63.5
190	"	"	"	0.105	"	36.0
				0.080	"	31.1
				0.040	"	14.1

The effective size was 0.033 mm.; uniformity coefficient, 5.1; albuminoid ammonia (parts in 100 000), 4.6; percentage loss on ignition, 5.6; weight per cubic foot, dry, 73.0; and percentage of voids, 54.4.

Separation in the mechanical analysis of the Somerset core was not carried beyond the 200-mesh sieve, or the 0.10-mm. size; and 94% of the material by weight was finer than this size. This is very fine material and perhaps may be in the 0.002-mm. class of cores mentioned by Mr. Hazen. Analysis of a sample of Englewood core—Miami Valley dam—gives 95% finer than 0.10 mm. and an effective size of 0.002 mm.

Compression and Seepage Tests on Core Material.

Many investigations have been made to determine the law followed by water in passing through a porous material. The earliest attempt was that of Darcy,* whose conclusion was that the rate of flow is directly proportional to the pressure, or head, and inversely proportional to the thickness of the material. That is, to double the head and keep the depth or thickness of the material the same, would double the flow, and to double the depth or thickness of the material and keep the head the same, would result in one-half the flow. This is known as Darcy's law. He also found that the flow varied with the diameter of the soil grains. The flow also varies with the porosity, which is not dependent on the diameter of the soil grains, but changes with the degree of compactness of the material.† Another important factor is the temperature—viscosity—of the water as it passes through the material.

That the rate of flow is not exactly proportional to the pressure—or head—but increases slightly with the head, resulting in a curve rather than a straight line, has been shown by King and others in a long series of laboratory experiments.‡ This, however, is a refinement not warranted in ordinary engineering practice.

The investigation of this subject by Mr. Hazen and his report§ thereon antedate those of Messrs. King and Slichter, and have become classic with American engineers. As a result of his experiments, Mr. Hazen found that the finer 10% of a material has as much effect on seepage rate as the coarser 90%, or, to state it in another manner, the material would have the same seepage rate it has, if the grains were all the same size as those dividing the 10% from the 90%. This size, than which one-tenth is finer and nine-tenths coarser, he calls the "effective size".

Another matter which he found to be important is the degree of uniformity in the sizes of the grains of a material, whether the grains are all of one size or of various sizes. This relation he expresses as a ratio between the size which has 60% finer than itself and that which has 10% finer than itself. This ratio he calls the "uniformity coefficient" and states that the data only justify the application of his formula to material having a uniformity coefficient below 5 and an effective size of grain of from 0.10 to 3.00 mm.

Although it may not be assumed that Mr. Hazen's formula is applicable to flows through the extremely fine materials used in earth dam cores, yet the mechanical determinations of effective size and coefficient of uniformity may be utilized as an index to the suitability of materials.

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* "Les Fontaines de la Ville de Dijon", par Henri Darcy, Paris, 1856.

† This is demonstrated theoretically by Slichter, 19th Annual Report of the U. S. Geological Survey, Part 11, 1899. See also, Water Supply and Irrigation Papers No. 67, U. S. Geological Survey, 1902.

‡ 19th Annual Report of the U. S. Geological Survey, Part 11, 1899.

§ "Some Physical Properties of Sands and Gravels, with Special Reference to Their Use in Filtration," by Allen Hazen, M. Am. Soc. C. E., Report, Massachusetts State Board of Health, 1892, p. 541.

As the coefficient approaches unity, a material of uniformly sized grains and higher percentage of voids is indicated; as it departs from unity a more dense material, having variously sized grains, is indicated.

For filters and filtration, a sand having a low uniformity coefficient and a high percentage of voids is required, while for concrete and the core of an earth dam, a dense material is sought.

For an earth dam core such a material is indicated by a low effective size and high uniformity coefficient, but what is still more important to secure impervious-

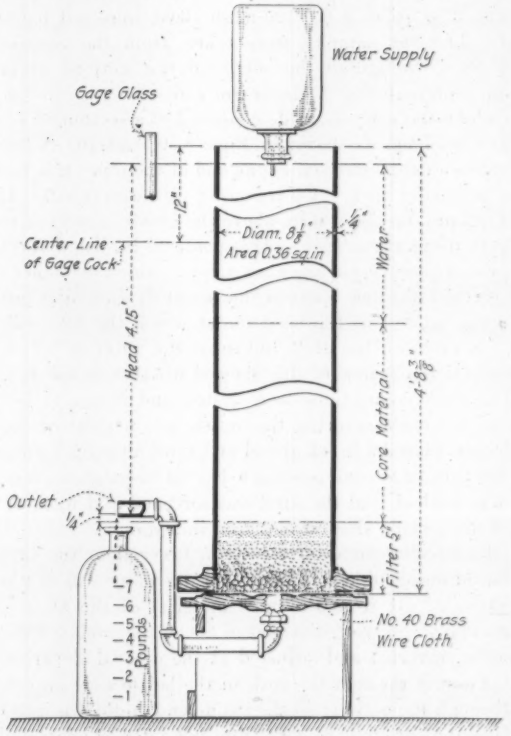

FIG. 8.

ness is the degree of compactness attained in the core itself. Even though a material is dense and fine, it is not impervious when first placed in the core. Until an over-burden has acted to compress a zone, the quantity of water that can pass through that zone is great, but as soon as pressure is applied the resulting compactness is rapid, amounting to 20% under a pressure of only 1 ton per sq. ft., and the reduction in seepage remarkable, being more than 90 per cent.

The methods followed and the formulas derived from the experiments mentioned will not be discussed here; these data are readily accessible and well known to engineers. There is no difficulty in applying these methods, but there is difficulty

in eliminating all the disturbing factors encountered in a series of experiments and a feeling, when one is done, that practice has not followed theory as closely as one could wish.

Compression and seepage experiments were made on samples of core material collected from two of the dams, as already described. These particular experiments were made to determine the effect of pressure of over-burden on the seepage rate through the material.

Paddy Creek Dam.—Of the 442 samples taken and screened, 266, or those containing more than 35% passing the 100-mesh sieve, were put together for making the experiment. The 266 samples were taken from the core on twelve cross-sections, five of them throughout the height of the sections, from bottom to top of dam, and were representative of the entire core.

The tank in which the test was made consisted of a section of 8-in. wrought-iron pipe about 4 ft. 8 in. long, as shown in Figs. 8, 9, and 10. A blank flange, with gasket, was bolted to a flange screwed to one end of the pipe; this formed the bottom of the tank, the top being open. A ⅜-in. outlet pipe connected at the center of the blank flange led out and up about 6 in. above the inside bottom of the tank and had a short, horizontal, discharge section. At a point 12 in. below the top and directly over the outlet, was a gauge-cock, the glass tube of which extended above the top of the tank. The vertical distance between the invert of the outlet pipe and the water surface in the gauge-glass was the head under which the flow took place and was measured here. A mark on the glass indicated the point at which the water surface should remain and a glance at this showed whether or not the head was being maintained. The tank connections were tested and found to be perfectly tight. On the bottom of the tank, covering the outlet, was a sheet of No. 40, brass-wire cloth and on this was placed 5 in. of gravel and sand filtering material, graded from 1-in. stones at the bottom to sand passing a No. 30 sieve, at the top.

The gravel was washed, and the sand was screened and washed, and screened a second time, and the varying sizes were placed in layers.

The top of the filtering material was ¼ in. lower than the invert of the short horizontal section of outlet pipe, and this formed a water seal to a point ¼ in. above the filter and excluded air from the open material of the filter. To maintain a uniform head a supply reservoir was placed above the tank. The "reservoir" was a 1-gal. glass bottle, inverted and adjusted at the desired elevation by means of a wooden frame. Passing through the cork in the bottle were two small glass tubes, one extending through the cork, from the inside end and down into the water of the tank, the other passing through the cork from the outer or lower end and extending into the bottle.

When the water surface in the tank had fallen slightly below the cork and end of the higher tube, air was admitted to the tube and passed into the bottle, displacing the water which, at first, flowed down the lower tube and then down both tubes, until another movement of air took place. This was repeated until the head was restored. A very slight drop in head would start the flow from the reservoir.

To prevent evaporation a small neck, small diameter, 1-gal. bottle was used to catch the seepage water. This bottle was "weighed in" on an accurate scale, and the pound points were marked on a strip of paper pasted to the bottle. The pound

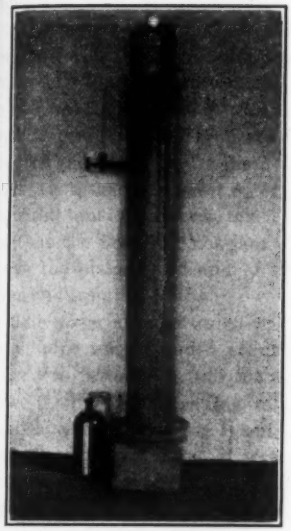

FIG. 9.—TEST TANK IN OPERATION.

FIG. 10.—MATERIAL UNDER PRESSURE IN TEST TANK.

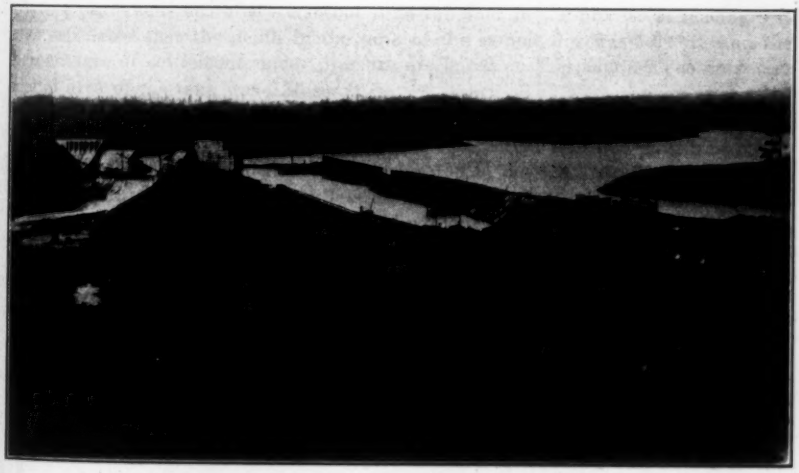

FIG. 11.—GENERAL VIEW OF CATAWBA RIVER DAM.

intervals were divided into tenths, thus permitting accurate readings of weight and volume of flow.

The temperature of the tank water was taken by a thermometer placed in the tank and resting on the sample surface. The thermometer was removed from the water only long enough to make a reading, that is, it was kept almost continuously in the water and directly on the surface of the material being tested. To induce settlement of the sample in the tank a piston and weighted levers were used. The head of the piston had a diameter of $\frac{3}{8}$ in. less than that of the tank, and to this head was attached a sheet of cardboard, $\frac{1}{4}$ in. thick and of a diameter slightly less than the tank. Care was taken to keep the piston rod vertical when in use and to see that the head did not jam. In calculating the pressures, the weight of water above the piston head and of the piston itself was added to that induced by the lever and weights.

The material to be tested was placed in the tank in the following manner: The outlet pipe was extended about 5 in. above the top of the filtering material and water was introduced into the tank and upward through the filter to a point 5 in. above it. The tank was then filled to within 1 ft. of the top by lowering water in a small bucket and tipping it out. A part of the fine core material to be tested was then placed in a bucket of water in the proportion of about 1 part to 3 parts of water, thoroughly mixed, and added to the water already in the tank. This was repeated three times, the water for the second and third mixings being taken from the tank and returned to it.

The mixings extended over two days, the material settling during the intervals, the object being to simulate sluicing into the pool of the dam. After the test had been completed and the core material taken from the tank, it was found that the coarser parts had separated somewhat from the finer in the process of placing. It was estimated that the depth in the tank of the sample dry was 1.84 ft., and the percentages of settlement under pressure are based on this depth. The cross-sectional area of the tank was 0.36 sq. ft.

The tank was first allowed to stand 9 days with the outlet closed and the tank filled to the top with water. At the end of this period the first measurement of depth of material was made, using a wooden rod to the end of which was attached a sheet of thin metal of slightly less diameter than the inside of the tank, thus covering the whole surface of the material without sinking into it. Following this, the plug was removed from the outlet, and the flow was measured for 7 days by the drop in the tank. The outlet was again plugged and allowed to remain so for 20 days, at the end of which time it was found that the sample had settled 0.151 ft. The plug was then removed and the flow was measured for 5 days. Up to this point in the experiment no pressure other than that due to the water had been applied to the material in the tank.

The seepage rate was high, amounting in 24 hours to several times the capacity of the supply reservoir. This flow was collected in an open bucket and returned to the tank and in this way the evaporation was determined and added to the measured drop to obtain the correct flow. Measurements were made at 9 A. M. and 9 P. M., to obtain both the day and night flow and the temperatures of the tank water were taken at the same time. Pressure was now applied to the sample by the piston, lever, and weights, the first load so applied amounting to 760 lb. per sq. ft. Following

each flow period, the load was doubled. During the fifth or last flow period the amount of seepage had become so small that it was measured only once in 24 hours. The measurement was made at 9 P. M. and the temperature of the tank water was taken at 9 A. M. and 9 P. M.

Whenever increased pressure was applied to the material in the tank an immediate settlement took place, which amounted to a greater part of the settlement induced by pressure. That is, most of the settlement took place quickly under each increased load and then slowly for the remainder of the period. In the dam the load is applied gradually and the core settlement is more uniform. In applying the load of 3 400 lb. per sq. ft., this initial movement was quite marked.

Pressure was maintained under each load until no further movement of the lever could be detected. Measurements were taken at the beginning and end of each flow period to determine whether or not the material in the tank had risen after pressure had been removed and during the flow period. No rise of material occurred during any of the flow periods. The pressure data obtained are given in Table 2, the pressures given during settlement and flow periods being those due to the column of water above the sample.

TABLE 2.

Number of days.	Period.	Pressure, in pounds per square foot.	Depth of material, in feet.	Total settlement.	Percentage of settlement of 1.84 ft.
9	Settlement.....	150	1.791	.....	....
7	Flow.....	120	.....	.....	....
20	Settlement.....	160	1.640	0.151	11
5	Flow.....	120	1.640	0.151	11
7	Pressure.....	760	1.505	0.286	18
12	Flow.....	170	1.505	0.286	18
12	Pressure	1 550	1.476	0.315	20
13	Flow.....	170	1.476	0.315	20
7	Pressure.....	3 400	1.411	0.380	23
12	Flow.....	170	1.411	0.380	23

Using the average flows under the varying pressures of the different periods, the maximum rate or quantity of water that will filter through the material with a loss of head equal to the depth of the sample, given in gallons per day per acre, is shown in Table 3. The quantities have been reduced to a uniform temperature of 50° Fahr.

TABLE 3.

Pressure, in pounds per square foot.	Seepage, in cubic feet per 24 hours on 0.36 sq. ft., with 4.15-ft. head.	Depth of material above outlet, in feet.	Seepage, in cubic feet per 24 hours on 1 sq. ft., with 4.15-ft. head.	Seepage, in cubic feet per 24 hours on 1 sq. ft., with 1:1 slope.	Maximum rate, in gallons per acre per 24 hours.
150	0.5721	1.770	1.5892	0.6778	220 800
160	0.5138	1.619	1.4272	0.5568	181 400
760	0.1210	1.484	0.3361	0.1202	39 200
1 550	0.0759	1.455	0.2108	0.0739	24 100
3 400	0.0185	1.390	0.1347	0.0451	14 700

Linville Dam.—An experiment similar to that described for the Paddy Creek Dam was also made with core material from the Linville Dam. Of the 577 samples

taken on five cross-sections, from the bottom of the dam up to 10 ft. below the flow line, 370 contained more than 35% passing the 100-mesh sieve. The 370 core samples were combined to make the test sample.

The same tank was used in this experiment as for the tests on the Paddy Creek Dam material. After placing the filter gravel in the bottom of the tank, water was introduced through the outlet pipe and upward through the filter to a depth of about 1 in. above the gravel.

Following this, the core material was put in the tank dry to avoid the separation that occurred when the Paddy Creek material was placed wet, and the sample was not rammed in the tank. It had a depth of 2.75 ft. in this loose condition.

Through an extension of the outlet pipe, water was again introduced into the tank and passed upward through the loose core material in a very short time, about 25 min. Pressure was applied to the sample at once, and the settlement and flow periods, as in the first part of the Paddy Creek Dam experiment, were omitted. Under a pressure of 710 lb. per sq. ft., this material settled 17%; the Paddy Creek Dam material settled 18% under a pressure of 760 lb. per sq. ft. The settlement data in Table 4 show a close agreement with the data given for the Paddy Creek Dam, in Table 2.

TABLE 4.

Number of days.	Pressure, in pounds per square foot.	Depth of material, in feet.	Total settlement.	Percentage of settlement of 2.75 ft.
11	710	2.281	0.469	17
14	1 470	2.240	0.510	19
11	3 360	2.125	0.625	23

Greater resistance to seepage is encountered by the water at the surface of the sample than lower down, and to remove or reduce this resistance, the surface of the sample was scratched. This was done with a knife and to a depth of about 1 in. The scratching caused a slight increase in depth of the sample and a considerable increase in the seepage rate. To show the effect of the scratching, the quantities were measured both before and after scratching the surface, and the results were not uniform, as may be noted in Table 5. In this table the rates have also been reduced to a uniform temperature of 50° Fahr.

TABLE 5.

Pressure, in pounds per square foot.	Seepage, in cubic feet per 24 hours, on 0.36 sq. ft., with 4.15-ft. head.	Depth of material above outlet, in feet.	Seepage, in cubic feet per 24 hours, on 1 sq. ft., with 4.15-ft. head.	Seepage, in cubic feet per 24 hours, on 1 sq. ft., with 1 : 1 slope.	Maximum rate, in gallons per acre per 24 hours.
710	0.1514	2.260	0.4205	0.2290	74 600
710	*0.1699	2.281	0.4719	0.2594	†84 500
1 470	0.1191	2.219	0.3308	0.1769	57 600
1 470	*0.1474	2.229	0.4094	0.2199	‡71 600
3 360	0.0835	2.104	0.2319	0.1176	38 300
3 360	*0.1072	2.110	0.2978	0.1514	§49 300

* After scratching surface of sample.
† 13% increase.
‡ 24% increase.
§ 29% increase.

The second or higher rate obtained in each of these test periods has been plotted on logarithmic paper and the resulting curve extended to greater pressures. Seepage data from Paddy Creek Dam and compression data from both dams are similarly plotted. (Fig. 12.)

If the pressures exerted by the overlying material at various depths in the dam were known, the corresponding seepage rate could be taken from the curve and the seepage determined for that particular depth and therefore for the full height of the dam. The Goldbeck pressure cells now installed in the Miami Dams are

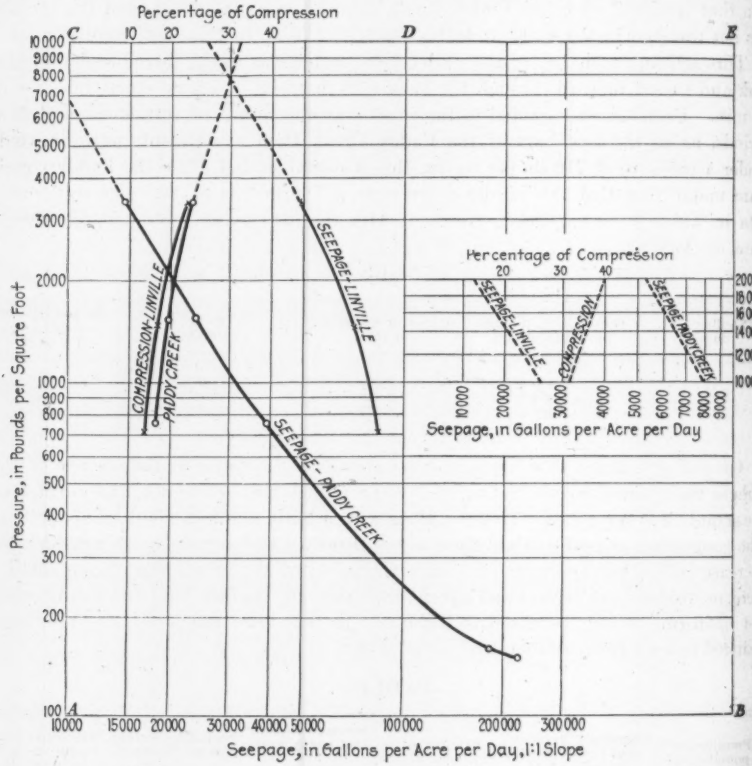

FIG. 12.

giving the information for those dams. Other valuable information needed is the weight per cubic foot and the percentage of contained water at various depths in the core. These will also be obtained at the Miami Dams by borings in the cores and from test wells in the masonry structures.

To estimate the total seepage through the Linville Dam, without knowing the weight of the material in the core or the depth at which a particular pressure takes place, the rate derived from the test under the highest pressure applied—3 360 lb. per sq. ft.—was used for the entire height of the dam; this gives excess seepage

for the lower portions. With water standing for a considerable time at full reservoir, the seepage was estimated to amount to 21 gal. per min., or 0.047 sec-ft. This is considering only that portion of the dam containing more than 35% passing a 100-mesh sieve or more than 35% fine.

Similarly, for Paddy Creek Dam the seepage was estimated to amount to about 10 gal. per min. This, however, was at a higher temperature and without having scratched the surface of the sample in the test tank. Reducing the temperature to 50° Fahr. would lower this estimate and scratching the surface of the sample would have raised it, but probably to an amount less than that estimated for the Linville Dam, if the scratching could have been to exactly the same depth and of the same amount. The scratching introduces a very disturbing element into the problem in that it cannot be done in a uniform manner.

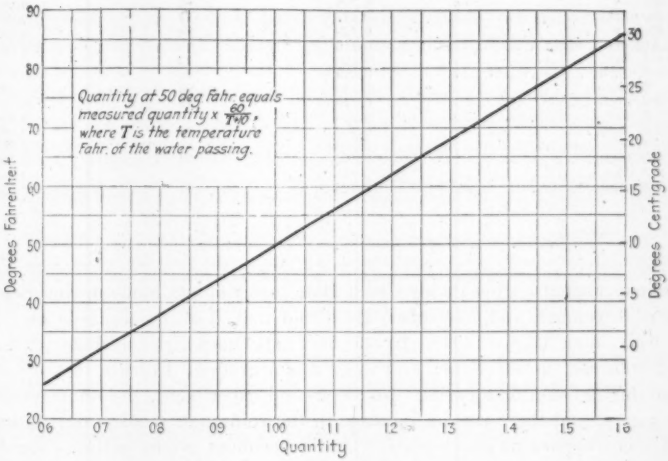

FIG. 13.

Careful measurements of depth of material were made at the beginning and end of each flow period, and no increase in depth occurred during any period, that is, there was no swelling of the material after pressure had been removed. The sample was found to contain 28.6% of moisture immediately after removal from the tank. It was very compact and was taken out with difficulty, and there was no stratification. This was the condition after a pressure of 11 days under 3 360 lb. per sq. ft. and a continuous flow of 21 days.

Temperature.—Viscosity.—Corrections from temperature of tank water, tailwater, or water in the gauge-glasses are all unsatisfactory in that they are not the same as that of the water as it passes through the sample. The temperature in the sample changes less rapidly than the water in the positions mentioned previously. The notes often show a greater flow for a low night temperature than for a higher day temperature. For a rising temperature, the correction is too large and, for a falling one, it is too small. To obtain results more nearly correct, more elaborate apparatus is necessary, and thermometers should be introduced into the

core material through holes drilled in the sides of the tank and fitted with stuffing-boxes.

Fig. 13 illustrates the importance of carefully obtaining correct temperatures and Table 6 shows how the record, made irregular by temperature changes (Columns 2 and 3), is smoothed out by applying the corrections (Column 4).

TABLE 6.

Date, 1919. (1)	Cubic feet per 24 hours on 0.36 sq. ft., with 4.15-ft. head. (2)	Temperature of tank water, in degrees, Fahrenheit. (3)	Cubic feet per 24 hours on 0.36 sq. ft., with 4.15-ft. head, reduced to 50° Fahr. (4)
July 30, day	0.128	85	0.081
" 30–31, night	0.117	77	0.081
" 31, day	0.128	84	0.082
" 31–August 1, night	0.119	79	0.080
August 1, day	0.128	83	0.083
" 1–2, night	0.119	79	0.080
" 2, day	0.130	83	0.084
" 2–3, night	0.119	75	0.084
" 3, day	0.123	78	0.084
" 3–4, night	0.113	70	0.085
" 4, day	0.132	84	0.084
" 4–5, night	0.122	74	0.087
" 5, day	0.137	86	0.086
" 5–6, night	0.123	77	0.088
Average	.....	....	0.0835

The experiment on the Paddy Creek Dam material was made during September, October, November, and December, 1918, and that from the Linville Dam during June, July, and August, 1919. In the first experiment, the average temperatures ranged from 60° to 66° Fahr., and there was a gradual decrease in the volume of seepage during each flow period. In the second experiment, the temperatures were much higher, and there was an increase in the volume of seepage during each flow period. The apparatus was housed. This experiment might well be extended in a laboratory where apparatus for applying greater pressures is available.

Some interesting experiments have been made by W. Spring on the permeability of clay.*

To determine whether or not clay would give up its moisture under pressure and in the presence of water, a mass containing 33.66% moisture was placed in a porous jar—battery cell—the cell surrounded by water, and the clay subjected to a pressure of about 3 tons per sq. ft. for 3 days. This reduced the moisture content to 26.86%, or 30.33% of the original quantity of water present.

A number of trials, using pastes containing varied quantities up to 70% of water and subjecting each to the same pressure, or 3 tons per sq. ft., resulted in reducing each sample to about the same degree of hardness and the same moisture content, namely, about 27 per cent. Doubling the pressure, or increasing it to 6 tons per sq. ft., further reduced the moisture to 23 per cent. Not much pressure is necessary to cause clay to give up its water, and a greater pressure makes a quicker expulsion.

* Spring, W., "Quelques Expériences sur la Perméabilité de Argile, Société Géologique de Belgique," *Annales*, Tome 28, pp. 117–127 (1901).

Another question investigated was: Is a clay which is mechanically prevented from expanding still permeable by water?

To test this, two porous earthen battery cells were filled with clay. The clay was dried, pulverized, sifted, and carefully tamped into the cells. Covers were bolted to the cells and through the cover of one was passed an open metal tube in which expansion might take place.

Both cells were placed in water, keeping the tube above the surface. The imprisoned clay absorbed only 3.37% of moisture and became very hard, while that in the open cell was soft and absorbed 12.09%, or nearly four times as much.

Other experiments were made with still greater care. Samples were placed in sections of an artery and imprisoned in porous earthenware. The function of the artery was to prevent solids, colloids, or substances dissolved in the water from passing into the sample.

An imprisoned sample of clay absorbed 2.67% of water, while a sample enclosed in an artery only absorbed 15.65%, or six times as much. Loam was found to take up 24.56% when firmly enclosed and 51.80% when free to expand.

In the case of a clay having a specific gravity of 2.62, the increase in volume was found to be almost proportional to the volume of water added. The contraction due to absorption of water amounts to less than 2 parts per 1 000 of the total volume, the conclusion being that, in general, a wet clay expands in proportion to the quantity of water it absorbs. The expansive force of clay in contact with water is estimated to be less than 2 kg. per sq. cm.

Vegetable Matter as it Affects Porosity.

The presence of vegetable matter in soil diminishes the porosity, and in proportion to the quantity of vegetable matter present. D. C. Henny, M. Am. Soc. C. E., found that surface soil containing vegetable matter had a seepage rate less than one-half that of volcanic ash, although the latter contained a much greater proportion of fine material.*

Very small percentages of vegetable matter have a notable effect on seepage rate, but just how these small percentages affect the flow is not known, although the theory has been advanced that it is by crushing into the voids. It is true that the percentage of vegetable matter is less by weight than by volume, but it does not seem to be enough to account for the remarkable decrease in flow through materials.

It is known that there is colloidal as well as vegetable matter present in soils. An examination of each size of grain of a material and a determination of the proportion of vegetable or colloidal, or vegetable and colloidal matter, accompanying each size and their effect on that particular size, would, in the writer's opinion, furnish valuable information as to the permeability of a material and disclose just how and why vegetable and colloidal matter influence the flow.

It has been suggested, in the case of colloids, that their characteristic swelling in the presence of water, and thus filling voids, might account for decreased porosity.

At the Somerset Dam the nature of the materials and the method of construction produced a satisfactory structure. The large quantity of rock in the dikes, due to the method of construction, made them stable, and their stability was

* *Engineering News*, Vol. 57, p. 251.

increased by sluicing from them the fine material which formed the core. At no time during construction were there indications of movement of the dikes. The same methods produced successful results with an entirely different material in the South, and in dams carried to a much greater height.

The methods being used on the Miami Valley dams in Ohio are also producing satisfactory structures; very clean gravel and sand are deposited in the dikes and the fine material is washed from them and into the cores.

All these dams, the Miami, the Bridgewater, and the Somerset, are characterized by thin cores and broad, stable dikes, but not all by fine core material. Those in the glacial region have fine material in their cores, but this is not so in the case of those in the non-glacial country. In the latter, however, experiments made with material taken from the cores show a low seepage rate.

During its construction period, a hydraulic-fill dam may be likened to a cantilever bridge in process of erection. Such a bridge will not again be subjected to the stresses that must be withstood during erection, nor will the dam, with its central pool, again be in so unstable a condition as it is in during construction. In designing an earth dam, this construction condition must be considered, just as it is necessary to provide for erection conditions in the case of a cantilever bridge.

DISCUSSION

ALLEN HAZEN,* M. AM. SOC. C. E. (by letter).—This paper is an important contribution to the solution of the problem of how small in grain size core material must be in order to be water-tight. For the materials described by Mr. Holmes, it amounts to a demonstration.

The laws governing the flow of water in filter sands and other coarse-grained materials are well known. With somewhat finer materials the same laws apply, at least approximately, but there may be a limit to the fineness of material to which they can be applied, and core material is so much finer than filter sand that a demonstration of the laws of flow in it is welcomed as a distinct addition to engineering knowledge.

In this case, the author has determined the grain size of the core material with the utmost care (the analyses having been made by Mr. H. W. Clark, who assisted in developing the methods at Lawrence, Mass., which are still in use), the material has been put in a testing device, and the rate of seepage through it under known conditions carefully obtained.

The effective size of the core material tested was found to be 0.033 mm. This is rather coarse, but, on the other hand, it is of the general degree of coarseness that must be used to permit drainage and consolidation in a large dam. The rate of seepage through it, when the head is equal to the depth of the material, is found to be about 8 in., or 200 mm., per day. Using the common formula for filter sand, with a coefficient of 800 for even-grained material, free from surface or other clogging, an expected rate of seepage of 870 mm. per day is computed. The observed rate of seepage is thus about one-fourth of the computed rate; or, putting it the other way, the actual rate of seepage is about that for a computed grain size of 0.016 mm., or one-half the actual size.

It should be noted in this connection that the determination of grain size for such small particles is by no means as accurate, relatively, as is the case with filter sands. Even with the most careful procedure, a considerable percentage error in the effective grain size of the core material must be expected.

These tests show that the use of the filter-sand formula for the flow of water in much finer materials will be at least somewhere in range; and as far as this test goes, the actual seepage is less than is indicated by the formula.

There is another interesting feature of these tests. After the first test the material was put under pressure to see how far it would be compacted, and its permeability was afterward again tested. Subsequently, greater pressures were applied and other tests made, until, finally, a maximum pressure, corresponding to a depth of fill of about 30 ft., was reached. It was found that each successive decrease in pressure reduced the volume of the material by a substantial quantity, and also decreased in much more rapid ratio the quantity of water that would pass through the material. In the last experiment, the flow was only one-fifteenth of what it was in the first one. This rapid decrease in porosity with pressure suggests that in the lower part of a high dam, with greater pressures than were used in the tests, almost complete water-tightness could be expected. It also suggests that

* New York City.

core material much coarser than that used in these tests might have been sufficiently water-tight in the lower part of the dam when subject to the pressure of the material above it.

It may be pointed out that to secure this compression and water-tightness it is essential that the material remain sufficiently pervious to allow the escape of the excess of water which must occur as consolidation proceeds, and that sufficient time must be allowed for the excluded water to make its escape. In the experimental apparatus, the greatest dimensions were only a few inches and this adjustment took place quickly. In the great mass of a large dam, the conditions of drainage would not be as favorable.

It may also be questioned whether all core materials would be compressed and tightened in this manner. For instance, if the core material consisted of rock flour, the grains of which were as hard as those of filter sand, it may be that, as with filter sand, a certain degree of consolidation would be quickly attained and that beyond that point increased pressure would scarcely reduce the volume or the permeability of the material. On the other hand, with a granular material so soft that the edges and corners would be broken by pressure, such a result as the author has found would be expected. No doubt many materials that will be encountered in dam construction would be capable of such compression as the author describes; but it may be questioned whether this result could be expected with fairly clean, hard-grained materials. At any rate, it would seem wiser not to count on obtaining it with a new material until the characteristics of that material had been definitely determined.

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Paper No. 1470

CREEPING OF RAILROAD RAILS*

By J. A. L. Waddell,† M. Am. Soc. C. E.

With Discussion by Messrs. Orlando Metcalf, C. A. Disbrow, C. D. Purdon, Leonard C. Jordan, Samuel T. Wagner, F. W. Green, Paul Hamilton, G. D. Snyder, D. L. Turner, Daniel Bontecou, B. H. Bryant, Louis C. Marburg, A. W. Buel, Billings Wilson, Herbert C. Keith, John Lundie, F. W. Skinner, James E. Howard, C. H. Chamberlin, Verne LeRoy Havens, William E. Robinson, Frank Reeves, W. M. Robinson, F. T. Darrow, Marcel K. Sessler, R. Emmet Keough, C. F. Yardley, Bernard Blum, and J. A. L. Waddell.

Synopsis.

In October, 1919, the writer was called in by some clients to investigate and report on the phenomenon of the creeping of railroad rails, in order to determine the cause thereof and, if possible, evolve a remedy. In agreeing to make the study, he stipulated that, after preparing his report, he should have the privilege of presenting his findings to the Engineering Profession; and, in accordance with that arrangement, he here submits the accumulated data to the American Society of Civil Engineers.

The first step taken was to send a letter, enclosing the following questionnaire, to the Federal Manager or the President of each of seventy of the principal railroad systems of the United States, Canada, and Mexico, asking for replies to the sixteen questions which it contained:

"1.—What troubles, in your experience, are engendered by the creeping of rails?

"2.—Is rail creeping proportional to the amount of traffic passing over a track?

"3.—With the traffic in one direction only, is the creeping greater on down grade or up grade, and to what extent?

"4.—On a single-track line, is the creeping greater in the direction of the heavy traffic; and, if so, to what extent?

"5.—On what portions of your line do you find the greatest creeping of rails—on tangents or on curves?

"6.—Does the degree of curvature affect the amount of creeping?

"7.—Does one rail in a track ever creep more than the other? Under what conditions?

"8.—For the same train loads, is the creeping greater on light rails or on heavy ones; and, if so, to what extent?

* Presented at the meeting of November 3d, 1920.

† New York City.

"9.—Does the amount of creeping vary with the nature of the roadbed, and with the kind and depth of the ballast; and, if so, to what extent?

"10.—Does the amount of creeping vary with the temperature and weather conditions?

"11.—Does the amount of creeping vary with the character and efficiency of the rail splicing; and, if so, to what extent?

"12.—Does the amount of creeping depend on the use or the non-use of rail-chairs or of tie-plates; and, if so, to what extent?

"13.—Have you found any satisfactory method of checking or of lessening creeping; and, if so, what is it?

"14.—Have you evolved any theory as to the cause or causes of rail creeping; and, if so, what is it?

"15.—Have you had any computations made as to the resultant force causing creeping under specific conditions?

"16.—Is there any information on the subject that you can furnish, not covered by the preceding questions; and, if so, what?"

The preceding request for information was honored by officials of 49 roads, or 70% of those addressed. This is, indeed, a truly gratifying result; and the writer, in addition to all previous acknowledgments, herewith tenders to each of those who so honored him his hearty thanks for the courtesy extended. In several cases opinions were given by a number of the officials of the system—in one case there were thirty-four reports from roadmasters; consequently, this paper is really a compendium of knowledge on the subject, furnished by more than one hundred of the best posted men in North America.

Those who answered the questionnaire represented, all told, 190 railroads or railroad systems, covering about 220 000 miles of line, because each Federal Manager generally had several systems under his control. This total length of line is 50% of the entire mileage of North America.

The railroads which replied to the questionnaire are the following, their arrangement in the list being alphabetical:

1.—Atchison, Topeka, and Santa Fe, *et al.*
2.—Atlanta, Birmingham, and Atlantic, *et al.*
3.—Atlantic Coast Line.
4.—Baltimore and Ohio, *et al.*
5.—Boston and Maine, *et al.*
6.—Canadian National, Eastern Lines.
7.—Canadian National, Western Lines.
8.—Canadian National, Government Lines.
9.—Central Railroad of Georgia.
10.—Central Railroad of New Jersey, *et al.*
11.—Chicago and Alton, *et al.*
12.—Chicago and Eastern Illinois, *et al.*
13.—Chicago and Northwestern.
14.—Chicago, Burlington, and Quincy, *et al.*
15.—Chicago, Milwaukee, and St. Paul.
16.—Chicago, St. Paul, Minneapolis, and Omaha, *et al.*
17.—Cleveland, Cincinnati, Chicago, and St. Louis, *et al.*
18.—Delaware and Hudson, *et al.*
19.—Denver and Rio Grande.
20.—El Paso and Southwestern.
21.—Erie, *et al.*
22.—Florida East Coast.
23.—Grand Trunk.

24.—Great Northern.
25.—Illinois Central, *et al.*
26.—International and Great Northern, *et al.*
27.—Lake Erie and Western, *et al.*
28.—Lehigh Valley, *et al.*
29.—Louisville and Nashville, *et al.*
30.—Michigan Central.
31.—Minneapolis and St. Louis.
32.—Missouri, Kansas, and Texas, *et al.*
33.—Missouri Pacific, *et al.*
34.—National Railways of Mexico.
35.—New York Central, *et al.*
36.—New York, New Haven, and Hartford, *et al.*
37.—New York, Ontario, and Western.
38.—Oregon-Washington Railroad and Navigation Co., *et al.*
39.—Pennsylvania Lines West of Pittsburgh, *et al.*
40.—Pere Marquette, *et al.*
41.—St. Louis-San Francisco, *et al.*
42.—St. Louis Southwestern, *et al.*
43.—Seaboard Air Line, *et al.*
44.—Southern.
45.—Southern Pacific.
46.—Southern Pacific Railroad Company of Mexico.
47.—Spokane, Portland, and Seattle, *et al.*
48.—Texas and Pacific, *et al.*
49.—Toledo and Ohio Central, *et al.*

In one particular the opinions of all railroad men on the creeping of rails are unanimous, *viz.*, that it is a phenomenon of serious import, very difficult to contend with, and involving both great danger to the traveling public and large expense to the railroad companies; consequently, the subject is one well worth investigating. The writer feels that the best he can do with the matter is to present a compendium of the replies given to the various set questions, offer a few suggestions of his own, and ask for a thorough discussion of the subject by railroad men in general, and all patentees of anti-creeping devices in particular. Some of those who answered the questionnaire may feel like elaborating their replies; and other railroad men who were not consulted, may have additional valuable information to offer. Quantitative records of rail creeping are still needed; and if any one possesses any, it is hoped that he will submit them in the discussion.

In order to facilitate the preparation of discussions, the writer has designated the opinions quoted by a combination of letters and numbers, the former relating to the reply itself, and the latter indicating the question discussed. For instance, *6-D* signifies that the statement which follows it gives the opinion of an unnamed railroad man, *D*, concerning Question No. 6. The writer has deemed it advisable not to mention the names of any of those whose opinions are quoted; hence, in discussing any point, instead of the usual method of arguing against the opinion of a certain individual, it will be necessary to deal with the statement itself and to refer to it by number and letter. If this method is strictly adhered to by all who discuss the paper, it will be easy for any reader of the discussions to locate the topic at issue.

The writer hopes that the discussion will be so full and complete as to cause inventors to try their hands at evolving some better anti-creeper than any which

has yet been used and which will solve in a truly satisfactory manner the problem of the absolute prevention of the creeping of railroad rails.

The various questions included in the questionnaire will now be stated, in due order, and will be followed immediately by a compendium of the replies thereto.

Question No. 1 and Résumé of Answers.

"What troubles, in your experience, are engendered by the creeping of rails?"

The direct or immediate results of rail creeping are as follows:

1-A.—Distortion of gauge, by reason of the slewing of the joints, widening it at joints on the high side of curves, and tightening it at joints on the low side thereof, with irregular line and gauge on tangents. This results in non-uniform gauge and surface, damage to rails, fastenings, and ties, and increased cost of maintenance. The widening of the gauge is likely to cause derailment, and its narrowing sets up a grinding action between rail-heads and wheel flanges, which is injurious to both.

1-B.—Bunching of ties, making it difficult to tamp efficiently and causing an unequal distribution of load from rails to ties. Such inequality causes a cutting of the timber, so that the full service life of the ties cannot be obtained.

1-C.—Splitting of joint ties.

1-D.—Closing of expansion joints at low points in the grade line and at turn-outs, crossings, movable spans, and other points where the creeping is obstructed. Such closure results in uneven gauge, and, during extremely high temperatures, sometimes in sun-kinks, and in throwing turn-outs, derails, crossings, and other track devices out of line or adjustment.

1-E.—Openings of joints near apices of grade, often resulting in breakage of track. Open joints are the cause of battered ends of rails.

1-F.—Churning of ties, causing depressions of track at joints.

1-G.—Sliding of ties, thus loosening the ballast.

1-H.—Bunching of ballast between ties and squeezing it outward, so that it is no longer truly efficient; also working the ballast from beneath the joint ties and leaving low joints. Unequal distribution of ballast is likely to result in bad drainage of roadbed.

1-I.—Crowding ahead of switch-points and frogs, necessitating in yards the continual moving forward, resetting, and adjusting of switch-points and head-blocks.

1-J.—Battering of ends of rails. This shortens the life of the rails, produces a rough-riding track, and causes unnecessary wear and tear, not only on all passing wheels, but also on the locomotives or cars to which they pertain.

1-K.—Kicking sideways of track. This is due to the closing of the spaces provided for expansion, followed by overheating of the rails, which can then no longer expand longitudinally. This condition is one of unstable equilibrium, and at any moment the track may spring sideways, but it generally requires some extraneous force to start such motion. That force is provided by the vibration of a passing train; and the sudden springing often results in derailment. It is difficult to take proper precautions against such accidents, because the heating of the rails is likely to occur quickly and the springing to happen without warning.

1-L.—Splitting of angle-bars.

1-M.—Loosening and pulling of spikes.

1-N.—Shearing of spike heads.

1-O.—Shearing of track bolts.

1-P.—Kinking of rails. When the track is held from springing sideways in great lengths, and when the expansion allowance has been exhausted, kinks are likely to occur in the rails, and it is almost impracticable thereafter to remove them.

1-Q.—Surface waves in rails. These are due primarily to unequal tie spacing, the churning of ties, and the sagging of track, all of which are mainly caused by rail creeping.

1-R.—Forming of wide gaps in track on muskegs, sometimes as wide as 18 in.

1-S.—Track carried ahead in summer, and joints pulling apart in winter. When the ties are well anchored to the rails, they are often drawn from their beds; and when they are not so anchored, a dangerous condition prevails, on account of the likelihood of buckling.

1-T.—Cutting of ties due to pulling of spikes and pounding of rails, thus letting water enter the timber and starting decay.

1-U.—Crooked track where the rail joints are staggered. This is avoidable, when any rail creeping exists, only by careful watching and constant adjustment. Rectification thereof is accomplished by the readjustment and respiking of joint ties, and, in some instances, by the cutting off or the driving back of steel.

1-V.—Interference with the operation of interlocking.

1-W.—Causing spring-rail frogs to stand open, and gaping of switch-points, thus inviting derailment.

1-X.—Weakening of the construction of frogs and switches.

1-Y.—Interference with the operation of dead rails on track scales and of the rails across draw-spans.

1-Z.—Finally, increasing materially all the difficulties known to trackmen.

This is a long list of troubles due to rail creeping, and many of the items are of serious import, involving, as they do, peril to human life, injury to trains, loss and damage of property carried, excessive wear of track and rolling stock, and great expense for maintenance and repairs; consequently, anything effective that can be done to lessen these troubles not only would be a real boon to humanity, but also would involve an economy of vast amount and world-wide importance.

Question No. 2 and Résumé of Answers.

"Is rail creeping proportional to the amount of traffic passing over a track?"

The vote on this question is 67 "yes" and 9 "no," 9 being uncertain. It is probable that the creeping is generally, but not always, approximately proportional to the amount of traffic; but, in many cases, other ruling factors intervene and prevent a fair conclusion from being drawn concerning this matter.

Question No. 3 and Résumé of Answers.

"With the traffic in one direction only, is the creeping greater on down grade or up grade, and to what extent?"

The vote on this question is 68 who state that the creeping is greater on the down grade, 2 who claim that it is greater on the up grade, and 3 who believe that there is no difference. Only 21 have given estimates of the extent to which

the creeping on down grades exceeds that on up grades, the amounts varying from 0 to 150%, with an average of 58 per cent. Of these 21, 13 estimate between 50 and 100%, only 1 passing the latter figure.

Many of the responders qualified their vote on this question, the most important of such observations being the following:

3-A.—The rails habitually creep down grade—accounted for, no doubt, by the faster speeds in that direction, and on heavy grades, in some degree, by the continuous application of brakes, and also by the back action of double-header engines. Locomotive operators on a descending grade habitually take a running start for an up grade; and the rails for some distance above the foot of the grade have been observed to creep up grade, due, of course, to the excessive speed of trains going in that direction. The tendency to movement by creeping due to the setting of brakes may have some influence on grades, where, no doubt, the retarded wheel has a tendency to skid the rail under itself.

3-B.—Creeping is greater on down grades, but the extent depends on the condition of the roadbed, the material in the embankment, and the solidity of the underlying ground.

3-C.—With traffic in one direction only, creeping precedes the traffic regardless of the gradient.

3-D.—With traffic in one direction, cases are known where the creeping has been greater on the down grade, but other instances are also known where the creeping has been greater on the up grade, this seemingly depending on the character of the roadbed and probably also on the relative speeds of the different kinds of traffic.

3-E.—Greater on down grade, extent depending on kind of equipment in which traffic is handled. Other things being equal, traffic handled in long cars will increase the creeping.

3-F.—Cases have been reported where the rails on the west side of the track crept in one direction and those on the east side thereof crept in the other.

3-G.—The extent depends on the gradient, its length, etc. With a certain stretch of 1 mile, the creeping is about one-third greater on the down grade than on the up grade under the same conditions.

From the preceding, it is evident that for traffic in one direction only, the creeping is almost always greater on down grades than on up grades; but that sometimes combinations of conditions may exist, which induce considerable creeping on the up grade.

Question No. 4 and Résumé of Answers.

"On a single-track line, is the creeping greater in the direction of the heavy traffic; and, if so, to what extent?"

The vote on this question is 75 stating that the creeping is greater in the direction of the heavy traffic, and 2 that it is not. Only 10 have given estimates of the extent to which heavy traffic augments creeping, as compared with light traffic, the amounts varying from 20 to 100%, and averaging 57.

The following observations were made qualifying some of the replies:

4-A.—Generally, yes; there are conditions obtaining, however, under which the rail creeps most in the direction of the lighter traffic, such as junctions of double and single track and water-stops or other stopping places where trains in one direction are given heavier braking applications than trains in the opposite direction.

4-B.—The extent is in proportion to the difference in weight of traffic. Actual experience shows that four heavy east-bound trains, with other agencies in effect, could drag steel 1 ft., while one west-bound train would drag it back 2 in. Other agencies which govern the extent of the creeping and the effects of which are indeterminable, as at times they vary under the same weight of traffic, are down grades, application of brakes, nature of roadbed, quantity and quality of ballast, and temperature.

4-C.—On this division the track has crept as much as 36 in. in six weeks. This was due to heavy traffic moving in one direction during the busy season.

4-D.—Yes; although, in swampy ground, creeping often occurs in both directions, changing with the passage of each train.

4-E.—Extent depends on grade and track conditions, difference in amount of light and heavy traffic, weight of cars, weight of rail, etc.

4-F.—On a single-track line, rail creeping occurs in the direction of the heaviest tonnage, fastest speed, and descending grades, the creeping under any of the conditions mentioned being aggravated by a soft or springy sub-grade.

4-G.—When the heavy traffic is up grade, the increased speed on descending grade, combined with the tendency to creep down grade, may balance or overbalance the tendency to creep in the direction of heavy traffic.

From the foregoing it may be concluded that, on a single-track line, the creeping is almost always greater in the direction of heavy traffic, any exception to this rule being due to an unusual combination of adverse conditions.

Question No. 5 and Résumé of Answers.

"On what portions of your line do you find the greatest creeping of rails—on tangents or on curves?"

The general opinion on this point appears to be much divided, the result of the ballot being as follows:

Greatest on tangent	37
Greatest on curves	10
No difference	25

Considerable explanatory matter was given in some of the replies, the most important being the following:

5-A.—Generally the difference is unnoticeable, owing to the fact that one rail sometimes creeps more readily than the other. This is due to the influence of the sun on the roadbed, making it softer on one side, in which case the curved track will show more creeping than that on tangents.

5-B.—On tangents. Creeping also takes place on curves, there being often a difference in the amounts of creeping between the inside and the outside rails. In some cases the inside rail on a curve creeps to a greater extent than the outside rail, and *vice versa.*

5-C.—Curves are generally tie-plated, and this eliminates the creeping to a certain extent.

5-D.—Greatest on tangents; but on curves of 2° or less there is no appreciable difference.

5-E.—As a general but not invariable experience, creeping of rails occurs chiefly on tangents on account of the fact that, ordinarily, the soft or muskeg ground, on which track creeps the most, is flat and, consequently, mostly of a tangent alignment.

5-F.—Rails very seldom start creeping on curves; for although the creeping there is often more pronounced, it is caused by the rails being crowded up from the tangent and thrown out of line on the curve.

5-G.—Creeping is greater on tangents, if the curves are properly elevated and properly operated. When they are improperly elevated, however, as is the usual case, and the greater weight in consequence is thrown on either rail, the rail receiving the preponderance of weight will creep the more.

5-H.—Four out of seven roadmasters state that the creeping is greater on tangents, and three are of the opposite opinion, all agreeing that there is very little difference.

5-I.—The greatest creeping of rail is usually found in dips at the bottom of long grades. In cases where curves occur on these grades, the inside rail seems to creep more than the outside one.

5-J.—Frequently an outside rail travels in one direction and the inside rail in the other. This condition likewise occurs on tangent track, but it has not been noticed so frequently.

The general deduction to be drawn from this mass of evidence is that there is, on the average, but little difference between the amounts of creeping on tangents and on curves, and that that which exists on the latter is usually carried over from the former and not started on itself, except when the super-elevation is improperly done.

Question No. 6 and Résumé of Answers.

"Does the degree of curvature affect the amount of creeping?"

The ballot on this question resulted in 34 affirmative and 29 negative votes; and from the numerous observations made the following are quoted:

6-A.—Amount of creeping is affected by curves, being less on high-degree curves than on low-degree curves.

6-B.—Yes, being more on heavy than on light curves.

6-C.—Only very slightly, if at all.

6-D.—The sharper the curve the less the creeping, for the reason that curves of more than 2° are ordinarily equipped with standard shoulder tie-plates. These tie-plates give the advantage of double spiking and eliminate the creeping to a certain extent.

6-E.—Variety of speed has more to do with rail creeping on curves (where the roadbed is right) than the degree of curvature. Slow, heavy traffic, passing over track that is elevated for high speed, has a tendency to crowd the inside rail down grade

6-F.—If the elevation and speed are properly adjusted and both rails evenly loaded, the creeping will be about the same as on tangent.

6-G.—The degree of curvature affects the amount of creeping, and the sharper the curve the less the creeping. Four roadmasters, however, oppose this opinion, stating that the degree of curvature does not affect the amount of creeping, while three others are equally emphatic in declaring the contrary. Local conditions may influence these various opinions; but they should not have any bearing on the general facts.

6-H.—Creeping is decidedly in some inverse proportion to rate of curvature.

6-I.—The greater the degree of curvature, the greater is the tendency to rail creeping, due to the greater friction between the wheels and the rails.

6-J.—On sharp curves no creeping is observed, unless they are on grades of more than 2 per cent.

6-K.—A greater amount of creeping occurs on stiff curves than on light ones, especially in the case of the inside rail.

6-L.—Creeping of rail is retarded by curvature; but where creeping occurs on curves, it is generally greater on the inside or low rail, due to the greater load on this rail from tonnage trains.

6-M.—There are several factors which determine the amount of creeping on curves—the proportion of weight distributed on the two rails; the speed of the trains; the manner in which the ties are spiked; the direction in which the train loads are heavier, if any, and the character of the grade or foundation for the track. On curves where the inside rail receives the heavier load, on account of a too great elevation of the outside rail, the inside rail creeps the faster. High speed, where the elevation is too small, will throw the greater portion of the weight on the outer rail; and the rail which receives the more weight, other conditions being equal, will creep the faster. Where there is no relative advance of either rail, it is quite likely that the effects produced by heavy, slow-speed freight trains and fast passenger trains are compensating.

The slipping effect of wheels on curves may also exert some influence on the relative amount of creeping of the two rails; but it is probable that the speed and super-elevation, which determine the distribution of the weight between the rails, are of more consequence. It is frequently reported that the outer or higher rail of a curve creeps faster than the inner rail.

The only certain deduction which can be made from the preceding evidence is that there exists a radical diversity of opinion on the subject. About one-half of the voters do not believe that the degree of curvature affects the creeping at all; and the other half, who think it does, are divided in opinion as to whether the creeping increases or decreases with the rate of curvature.

Question No. 7 and Résumé of Answers.

"Does one rail in a track ever creep more than the other? Under what conditions?"

The vote on this question is almost unanimous, being 66 in the affirmative and only 5 in the negative—and the latter somewhat uncertain. Many opinions as

to causes, conditions, etc., have been given, from which the following have been excerpted:

7-*A*.—Yes; when one side of the roadbed is softer than the other, due to shifting the line partly off the old roadbed, and in locations where, due to the topography of the country and the peculiarities of the alignment, one rail stays longer in the sunshine than the other.

7-*B*.—In most instances one rail creeps more than the other. Many conditions may account for this, such as inequality of joint efficiency, temperature, small holding power of spikes at either end of the tie, due to decay from weather conditions that affect one side of the track more than the other, and unequal tamping.

7-*C*.—In one instance, on a 4-mile stretch of track where the curvature was about balanced and where both rails were creeping, one rail gained about 3 ft. on the other.

7-*D*.—In some cases, and not infrequently, when the inner rail creeps more than the outer one, the explanation therefor seems to be the heavier loads borne by the former, due to the elevation given for passenger-train speeds.

7-*E*.—On tangent, creeping is equal in both rails, provided they are equally well fastened, except where continuity is broken on one side by switches; but on curves the low, or inside, rail creeps more than the outer one.

7-*F*.—Where curves have a super-elevation to suit a speed of 25 miles per hour, a heavy coal train moving 12 miles per hour will throw more than half its load on the low rail.

7-*G*.—One rail creeps more than the other, especially on the south side of the track, when the frost is coming out of the ground in the spring.

7-*H*.—Rail does not creep more on one side than the other, if both are equally well fastened, except where continuity is broken on one side by switches.

7-*I*.—The high rail on curves creeps more than the low one in the average case, as the super-elevation is usually insufficient for the speed used. On tangent, if one rail is out of surface and higher than the other, the high rail will creep the more. The closer the track tallies with correctness as to line, surface, and composition of sub-grade, the closer the similarity will be in the creeping.

7-*J*.—Yes. Generally this can be traced to a difference in temperature of the two rails, usually encountered on hillside track where one rail is thrown into shadow in hot weather earlier than the opposite rail.

7-*K*.—It has been found that the creeping of the north rail and the west rail is greater than that of the south rail and the east rail, respectively. It has been observed also that the unequal creeping of rails is more emphasized on down grades than on up grades.

7-*L*.—Where one rail is unbroken for a long distance by a frog or switch, it will creep more than an opposite rail which is broken in one or two places by turnouts, cross-overs, etc.; also, one rail creeps more than the other in places where no satisfactory explanation can be offered.

7-*M*.—Yes; and more on grades not exceeding 1.5%, where the trains have a velocity of more than 25 miles per hour.

7-*N*.—Unequal creeping occurs on up grades where pusher-engine service is used.

7-O.—Grating of engine wheels and car wheels on the curves will cause one rail to creep more than the other.

From the preceding it is evident that one rail does creep more than the other and that this generally occurs, also that there is a variety of causes to account for the phenomenon. There is a difference of opinion as to which of the two rails on a curve moves the more, but the preponderance of the evidence shows that it is the inner one.

QUESTION NO. 8 AND RÉSUMÉ OF ANSWERS.

"For the same train loads, is the creeping greater on light rails or on heavy ones; and, if so, to what extent?"

The result of the ballot is that 74 state that the creeping is greater for light sections, 2 claim the contrary, and 4 believe there is no difference. The following are the most important of the observations recorded:

8-A.—Creeping is greatest on 85-lb. rail, with but little trouble on lighter rail. The greater creeping of the heavy rail is attributed to the greater traffic that it carries.

8-B.—Where 60-lb. steel has been replaced by 80-lb. steel, the creeping has been reduced under about the same traffic conditions.

8-C.—For the same traffic, the creeping is very much greater on light steel than on heavy—frequently where light rail has been creeping and has been relaid with heavier steel, the creeping has been largely, if not entirely, eliminated.

8-D.—Where the steel has crept 18 in. per month on 60-lb. rails, there was no evidence of creeping on 85-lb. steel adjoining.

8-E.—Would estimate that creeping is 75% greater on 60-lb. rail than on 80-lb.

8-F.—Greater on light steel by 100 per cent.

8-G.—Creeping is greater on light rails to at least three times the extent noted on heavy rails.

8-H.—The lighter the steel the greater the tendency to creep, owing to the wave-like motion caused by the slight upward and then downward movement of the rails and ties just preceding a moving train or locomotive. It has been noticed that the wave motion of 56-lb. steel is greater than that of 60-lb. steel, and that the 56-lb. rail will creep faster than the 60-lb. rail.

8-I.—There is a great difference between the creeping of light steel and heavy steel; for instance, after changing from 60-lb. to 80-lb. rails, no difficulty was ever afterward experienced. This is due to the facts that the heavy steel has more resisting power, and that there are eight slotted holes in angle-bars on 80-lb. or 85-lb. rails, whereas light rails have only four slotted holes.

8-J.—Creeping on track having light rail sections is sometimes entirely overcome by replacement with rail of 12-lb. to 20-lb. heavier section.

8-K.—For the same train loads, the creeping is greater on light rails than on heavy ones; and it seems to be proportionate to the difference in weight of rail. This is undoubtedly due to the fact that the heavier the rail the greater the expansion. One roadmaster reports that a 50-lb. rail will expand ½ in., while a 90-lb. rail will expand ¼ in., under the same temperature variation.

8-L.—It does happen frequently, however, that, in relaying 63-lb. or 74-lb. rail with 90-lb. rail, creeping will show up where it had not before been noticed.

8-M.—Usually. However, heavy rail generally goes with better maintenance, ballast, ties, fastenings, etc.; hence comparison would be somewhat difficult on account of all the other conditions not being equivalent.

8-N.—On rails of the same weight and different sections, the wider the head the greater the creeping.

From the preceding evidence, it may be concluded that light rails creep much more than heavy ones, but the rate of variation in creeping with weight of rail is uncertain. The large variations recorded by some observers are probably not due so much to greater wave action as to variations in the quality of the track.

Question No. 9 and Résumé of Answers.

"Does the amount of creeping vary with the nature of the roadbed, and with the kind and depth of the ballast; and, if so, to what extent?"

The ballot on this question indicates that 76 voters are of the opinion that the creeping is influenced by the character of the roadbed and by the kind and depth of the ballast, while only 4 hold the contrary belief.

The principal individual views as to causes and extent of the creeping in connection with these conditions are as follows:

9-A.—Amount of creeping varies with nature of roadbed, being greater on light than on heavy soils, and is affected very decidedly by the ballast. Track having not less than 12 in. of rock ballast under ties is seldom seriously affected by rail creeping.

9-B.—The amount of creeping varies with the nature of the roadbed and also with the kind of ballast. The rate of creeping is greater where the surface is of unsound soils than where it is of compact soils. The creeping in sand ballast is very much greater than that in stone ballast, because of the greater ease of movement of the cross-ties in the former.

9-C.—The higher the standard of maintenance of a railroad the less the creeping. Where the roadbed is composed of light materials and where the drainage is poor, more rail creeping is experienced than where the roadbed is of heavy materials and where good drainage prevails.

Proper depth and kind of ballast lessen creeping, as better drainage is provided, and as train loads are better distributed over the roadbed, thus giving a more rigid track structure and thereby lessening the wave motion in the track brought about by moving trains.

9-D.—It is generally greater on soft, boggy ground and in fine ballast, the particles of which are round rather than angular. There are cases where track has crept in one direction 5 ft. in ¾ mile during the seven months in which the roadbed is not frozen. This is an exceptional case, where the railway crosses a swamp.

9-E.—Creeping is generally confined to places where the bottom is soft; and it is aggravated by the embankments being constructed of a soft material from the adjoining ground, also by the tendency of the ballast to sink and disappear in an embankment of this nature.

9-F.—Where coarse ballast was used, it helped to counteract the creeping of the ties; and where the track is lightly ballasted, creeping takes place to a greater extent.

9-G.—On a roadbed of springy muskeg or the like, creeping is almost invariably present. To offset it under such circumstances, the general remedy is to lift the track with ballast, and, if possible, drain the wet ground. In some cases cinder ballast to a depth of several feet has effected a cure; and in other cases heavy gravel ballast of a like depth has proved to be the solution of the trouble.

9-H.—Where steel is properly jointed and fully spliced, it naturally creeps more in fine ballast than in coarse. With an 18-in. thickness thereof, the creeping will be twice as great in the former as in the latter.

9-I.—Creeping has been almost entirely overcome by increasing the strength of the roadbed. This is not always accomplished by the quantity of material used. More depends on the nature of the material. Experience shows that ballast, as such, will not stop creeping.

9-J.—The more solid the roadbed the less the creeping, regardless of whether it is caused by the material forming the roadbed, the material forming the ballast, or the condition of the cross-ties. As an illustration—there is very little creeping ordinarily on any track, other than that over marshes, when the roadbed is frozen solid in the winter.

9-K.—When the track is well ballasted with crushed rock, joints tight, and well tied and spiked, the creeping will be less than under other conditions.

9-L.—On a recently worked roadbed, that is, one where an unusual number of new ties have been installed, and where, consequently, resurfacing and lining have had to be done, the creeping will be much greater than on an old roadbed.

9-M.—Yes. This is especially true with tie-plated track.

9-N.—Creeping is greater in freshly laid ballast than after the ballast has settled and solidified. Creeping is greater on embankments than in cuts.

9-O.—On some stretches of dirt track creeping does not exist, because of the light traffic. On the other hand, the greatest creeping is on the best ballasted track, because that is where the most traffic is carried.

9-P.—Where stone ballast is used, the creeping is influenced by the depth thereof; when the track has 7 or 8 in. of stone under the ties, the creeping is not nearly so noticeable as on track with less ballast.

From the great mass of information furnished on this question, the conclusion is unavoidable that the amount of creeping does vary with the nature of the roadbed and with the kind and depth of the ballast. It is shown also that on embankments through swamps and muskegs, the creeping is more accentuated than it is anywhere else—also that the better and thicker the ballast, the less will be the amount of the creeping.

Question No. 10 and Résumé of Answers.

"Does the amount of creeping vary with the temperature and weather conditions?"

The result of the voting is that 75 state that the temperature and the weather conditions do affect the amount of creeping and 10 say that they do not. The following are the most important of the observations offered:

10-A.—Rail creeping is affected by temperature, being greater when there is considerable change therein between morning and night. It is also affected by weather conditions, being greater in wet than in dry seasons.

10-B.—Creeping is influenced by temperature, especially in the north when a great deal of expansion and contraction is taking place each day due to the wide range of temperature between the hot days and the cool nights.

10-C.—During the winter months the creeping is less, due to the ties becoming anchored in the ballast, also because the embankment is firmer in a frozen condition, the wave motion of the steel under passing trains being greatly reduced.

10-D.—The trouble from creeping track is greatest in the spring of the year just after the frost is out of the ground and when the sun begins to get very warm. Again, in the fall of the year, just before freeze-up, creeping is more noticeable than it is in the extreme hot weather of the summer or the cold weather of the winter.

10-E.—In the summer, and especially in the fall, rails are likely to creep into sags during the very cool nights when contraction is greatest, followed by extremely hot days. Then the rails either shove ahead or buckle. This is more noticeable in cuts where the weather extremes are exaggerated and where the heat is more intense.

10-F.—The rails will move faster over the ties in winter unless tie-plates are used, owing to the ice which forms under the rail and gives it a slippery surface on which to slide. In winter, by contraction, and in summer, by expansion, bolts are snapped, angle-bars broken, rails sun-kinked, and gauges widened.

In spite of ten opinions to the contrary, the preponderance of evidence is to the effect that both the temperature and the weather conditions in general have considerable to do with the amount of creeping. Similarly, it is generally recognized that in summer creeping is more active than it is in winter.

Question No. 11 and Résumé of Answers.

"Does the amount of creeping vary with the character and efficiency of the rail splicing; and, if so, to what extent?"

The ballot indicates that 75 are of the opinion that the amount of creeping does vary with the character and efficiency of the rail splicing; but 12 are of the contrary opinion.

The following are some of the individual reasons governing the answers to the question propounded:

11-A.—The creeping seems to vary with the character and efficiency of the rail anchorage, rather than with the character and efficiency of the rail splices.

11-B.—With a heavy, high-carbon splice-bar, and with track fully tie-plated, the track structure becomes more rigid, thus reducing the undulations of the rails and thereby allowing less creeping than with a light section of bar and no tie-plates.

11-C.—The rail splicing, considered as splicing only, has no effect on the creeping of rails. The modern splices always have spike slots, which, when spiked, tend to hold the rail in position and prevent creeping. Some of the older joints do not have these slots, and, therefore, allow the rail to run freely.

11-D.—Yes; to a small extent, but a rail joint is not sufficient in itself to check creeping, however efficient it may be. Good joints with deep, double slots in the angle-bars assist; but after a time, under the constant influence of heavy traffic, the slots in the angle-bars become worn and disengaged from the spikes, or the spikes very often break off, due to over-stress. Where tie-plates are in use this is more likely to occur.

11-E.—Seldom have much trouble with creeping rail on new track. This is because the rail splicings and the rail fastenings are all snug and tight; and, unless the roadbed is very soft and springy, there is not much danger of the rails creeping.

11-F.—Yes, greatly. The double-slotted angle-bar is a great improvement on the old style. On track where every rail fastening, bolt, and spike are all doing their work, there will be, practically speaking, under our conditions, no creeping. Such detail work is of the utmost importance under steadily increasing traffic conditions; and this is one of the hardest things to educate some of the track foremen to realize.

11-G.—Splices of the Bonzano type which have the flanges bent down between the ties are the best kind of splice to use, especially with long ties. They keep the ties from slewing and stiffen the joints, the latter, as a rule, being the most flexible part of the rail.

11-H.—There is a decided difference of opinion among the roadmasters in regard to the wisdom of spiking the rail in the slots provided for that purpose in angle-bars. Five of them state that, if the rail splicing has no slot holes for spiking, the creeping would be 50% greater, while two of them are absolutely opposed to the spiking of splices through slot holes and maintain that no slot holes should be provided. This difference of opinion is caused entirely by the difference of conditions in respect to the territory of which these roadmasters have charge. On the first-class track, where rail of not less than 80 lb., and up to 100 lb., per yd. is used, and the ballast is of sufficient depth and quality, and where the roadbed is of equally good and first-class consistency, it is a decided advantage to have rail splices provided with slots for splicing, while the contrary is the case on lines of lesser importance, and where the conditions as to first-class track are not entirely fulfilled. Track joints must be given preference attention on all matters of track; and the keeping up of the efficiency of the rail splicing forms an important factor in the prevention of creeping.

11-I.—The creeping of rail is affected by rail joint or splice only as the joint is properly or improperly maintained.

11-J.—Rail joints which are a combination of splice and rail-anchor tend greatly to reduce creeping.

Notwithstanding the fact that a number of those consulted believe that the character and efficiency of the rail splicing have no effect on rail creeping, so

many more are firmly of the contrary opinion, and give such good reasons in support of their claims, that it is impracticable to do aught else than agree with the decision of the large majority. As pointed out by one of the contributors, what may be true at one place or under one set of governing conditions may not hold good elsewhere; and it is probable that this is the cause of the decided diversity of opinion.

Question No. 12 and Résumé of Answers.

"Does the amount of creeping depend on the use or the non-use of rail-chairs or of tie-plates; and, if so, to what extent?"

Concerning the effect of tie-plates on the amount of rail creeping, there appears to be a wide diversity of opinion. Of the replies, 51 state that the tie-plates do affect it and 25 that they do not; but among the former there is a serious disagreement, for some say that the tie-plates decrease the creeping and others that they increase it.

The following are the most important of the observations on this point made by the responders to the questionnaire:

12-A.—Rail-chairs or tie-plates do not prevent creeping, except those tie-plates which are combined tie-plates and rail-anchors.

12-B.—Where tie-plates are used, the ties do not become rail worn, and tighter spiking can be maintained; which conditions prevent, to a great extent, the sliding of the rail over the ties. Each tie, therefore, has a better chance to act as an anchor.

12-C.—Tie-plates do not affect the creeping, except that they prevent the spikes from being forced out of the angle-bar slots at the joints; but, in the case of excessive creeping, the spikes will shear off, or the ties will roll and pull them out.

12-D.—Yes; to a very great extent. Tie-plates properly placed and fully spiked, with the spikes always kept tapped down close to the base of the rail, are a great preventive.

12-E.—Tie-plates are a valuable asset to overcome running steel, especially the Lundie tie-plate, which has a continuous shoulder, also two holes on each side. With these tie-plates in use and two spikes on each side driven down to the base of the rail at all times, there is very little chance of creeping.

12-F.—Tie-plates prevent the rails from cutting the ties; and, as this effect impedes the vertical movement of the rail, it helps to retard the creeping. They also restrain the formation of ice under the base of the rail, which ice is one cause of spread track and running steel. When fully spiked with the spikes kept driven down to the rail, they are a valuable asset to check creeping.

12-G.—Creeping of rails is increased by the use of tie-plates to the extent that they provide easier movement of the rail.

12-H.—Three roadmasters maintain that the presence of tie-plates will cause more creeping of rail by 30% than on track without tie-plates, while three of them maintain that the presence of the tie-plates has no bearing on the question, and that they will not increase rail creeping. One roadmaster even went so far as to state that the creeping is less on track supplied with tie-plates than it is on track without them.

It is significant to note that the three roadmasters first mentioned have charge of the best lines, most of which are double track and consist of first-class construction throughout, while the three claiming that there is no difference have charge of territory of lesser importance and a consequent lower quality of track. The last named has charge of territory of very unimportant character—mostly branch lines with little or no ballast. The conclusion drawn from these opinions, based also on experience, is that, on track constructed and maintained in first-class shape, tie-plates will cause a great deal more creeping than is the case without them.

12-I.—It appears that tie-plates facilitate slightly the movement of the rails.

12-J.—It was found that where trouble was experienced with creeping before tie-plates were used, this trouble was greatly reduced after tie-plates were applied.

12-K.—The use of tie-plates has a tendency to reduce creeping.

12-L.—If no tie-plates are used, after a short time the spikes will offer absolutely no resistance whatever to the rail.

12-M.—If the spikes are not kept driven tight, the use of tie-plates will greatly increase the creeping.

The preceding evidence goes to show that tie-plates without lugs increase rather than diminish the creeping; but that when the track is kept in good condition this detriment is small. Tie-plates are a necessity for first-class roads carrying heavy traffic, and no one would consider for an instant abandoning their use on the plea that they sometimes facilitate the creeping of rails.

Question No. 13 and Résumé of Answers.

"Have you found any satisfactory method of checking or of lessening creeping; and, if so, what is it?"

13-A.—Generally, good roadbed and track construction with a high degree of maintenance; directly, anti-creepers or rail-anchors.

13-B.—The only satisfactory method of checking creeping, in addition to inserting spikes through the slots cut in the angle-bars, is the placing of rail-anchors so that the thrust of the creeping rail is brought to bear against cross-ties in addition to the joint ties.

13-C.—Creeping can be reduced to a considerable extent by the use of rail-anchors, placed on the rail base opposite the joints and on the quarters and eighths of the rail—also by bolting two angle-bars through the web of the rail and blocking between the ties with wooden check-blocks.

13-D.—Especially-long ties have been found to be satisfactory. The Rail-Anchor tie-plate is one of the most efficient devices. To obtain satisfactory results with anchored rails, too close contact with adjoining rails should be avoided.

13-E.—Improve, if possible, the drainage conditions. Widen the shoulders of the embankment where necessary. Improve ballast conditions. Relay with heavier steel. Use tie-plates, if not on every tie, then, say, on every other tie, or on every third or fourth tie. If the rails are laid with broken joints, additional angle-bars can be put on at the centers of the rails opposite the joints. This will greatly resist creeping. Any improvement which adds to the stability of the roadbed or track will help to overcome the tendency of the steel to creep.

13-F.—Where the track passes through muskegs, swamps, or other places that are so soft as to cause a springy or wavy motion, the only way to overcome the trouble is by ditching, draining, rip-rapping the dump, and using heavy ballast, good ties, and heavy rail well fastened. The main thing is to start from the bottom and build up; for, unless the roadbed is first improved in such places, no amount of rail fastenings or anti-creepers will suffice.

13-G.—Creeping can be stopped by sufficient ballast, good joints with deep slots (or, preferably, Bonzano joints), and properly constructed rail-anchors, using 12-ft. ties at joints.

13-H.—Applying a pair of angle-bars to the center of the rail and using five rail-anchors per rail will overcome creeping to a great extent—in fact, will almost entirely prevent it, except on very soft muskeg track.

13-I.—Tie-plates and anchors in sufficient quantity check creeping.

13-J.—Yes; build up a shoulder at least 4 ft. from each rail and bury the ties to full depth, leaving only the top of the tie exposed for its full length, and give a drop of 4 in. from the ends of the ties to the edge of the slope.

13-K.—Yes; by spiking the ties so that the outside spike will be ahead of the inside spike in the direction of the creeping movement. Also, by cutting wooden blocks and putting them between the ties under the rails for several ties ahead of the joints in the direction of the creeping.

13-L.—Under ordinary conditions, if one-third of the ties are used to resist the creeping tendency, the resistance of the ballast against the thrusting force of the creeping is sufficient.

13-M.—Experimented a great deal and very satisfactorily with rail-anchors for the purpose of lessening or stopping the creeping of rails. The experiments covered the various rail-anchors that are on the market, and the Vaughn was found to be the most satisfactory, using in extreme cases as many as eight anchors per rail.

13-N.—There are many types of rail-anchors or anti-rail creepers on the market; and these devices, if properly used, will check and altogether do away with rail creeping. The most successful types are those built on the wedge principle, where the pressure against the tie results in tightening the wedge.

13-O.—No satisfactory method to check creeping has been found. The present anti-creepers are only partly efficient.

13-P.—The P-M anti-creeper has been used since 1907, and has given the best results. The Vaughn anti-creeper and the Henggi rail-anchor have also given good service.

13-Q.—The application of some of the better types of rail-anchors is desirable where the creeping is bad; but none of them is perfect, nor is any method known that is entirely satisfactory in overcoming creeping.

13-R.—As far as possible, remove conditions which probably are responsible for creeping rails; re-space the rails so that each joint has the proper expansion allowance; tighten the joint bolts uniformly; space the track ties to fit the joints so that they will afford uniform bearing throughout the rail length; tamp the ties carefully and see that the spaces between them and at the ends are well filled with ballast to hold the ties in position; and apply anti-creepers at the joint ties, at

the quarter points in the rail length, and at as many additional points as may be necessary to hold each rail in its proper position.

Question No. 14 and Résumé of Answers.

"Have you evolved any theory as to the cause or causes of rail creeping; and, if so, what is it?"

14-A.—Creeping of rail is due principally to the wave motion of the rail, track, or roadbed under traffic.

14-B.—Movement of rails is accelerated by an unsound roadway, and also by the driving force of the traffic acting against the rail joints and at other irregular places in the surface of the rail.

14-C.—Rail creeping is attributed to a number of causes: First, temperature stresses combined with a one-way movement; second, wave motion in the rail—with traffic in one direction only, train wheels are always running up hill on the rail, tending to push it ahead; third, braking stresses, the drag of the wheels over the rail combined with the wave motion; also the pushing or dragging force exerted by wheel flanges in contact with the ball of the rail on curved track.

14-D.—Creeping track is generally caused by soft dumps or roadbed.

14-E.—Creeping is also caused by unbalanced traffic in one direction, and by the contraction and expansion incident to changes in temperature.

14-F.—A yielding foundation, such as is present on muskeg track. A longitudinal movement of rail of 13 in. on account of the passage thereover of about forty-five cars of grain has been measured. The rail in this case was of 50-lb. weight; and the muskeg in question was one of the most severe for creeping of track.

14-G.—Track always creeps in muskegs, as the engine is pushing the track ahead. It is like rolling a heavy barrel over a feather bed—there is always a pile of feathers ahead of the barrel.

14-H.—The theory is proposed that, in most cases, creeping is due to the ballast being of too sandy a formation and the rails bending under heavy loads, with the wheels, as it were, always climbing a grade and shoving the steel ahead of them.

14-I.—It is a difficult matter to evolve a theory as to the causes of rail creeping. On double track, where traffic is all in one direction, rails creep with the current of traffic, whether on up grade or on down grade. On single track, where the roadbed is built on marshy ground, the track creeps with the current of traffic in either direction. Also, where traffic is equal in both directions, track will creep under the action of one train from 2 to 3 in., and a train in the opposite direction will reverse the creeping for the same distance; in fact, where traffic in both directions is equal, the creeping is compensated, and the resultant approaches zero.

14-J.—Unstable roadbed, light rails, excess of traffic in one direction, and use of long cars, the latter being the principal cause. Experience has shown that, even where roadbed is built across a swamp and where exceedingly heavy traffic is handled, creeping is almost unknown when the traffic is carried in short cars, such as those used for transporting iron ore, the length of which ranges between 25 and 30 ft. The same traffic handled in cars 40 ft. long increases creeping to

a very great extent, since under the long car the rail wave is much greater than under a short car.

14-K.—Some of the more important causes of rail creeping are: First, on down grades the action of gravity, together with the effect of applying brakes; second, rails laid too closely, no proper allowance being made for expansion and contraction; third, ties not properly buried in ballast so as to prevent their shifting; fourth, joint bolts not kept tight; fifth, wave action of rail under traffic.

14-L.—Creeping is caused by the roadbed absorbing the momentum of moving loads when speed is retarded; the contraction and expansion of bolts that are too tight or too loose; expansion allowance of rail being too great or unevenly distributed; soft roadbed; insufficient ballast below or between ties; friction caused by skidding of wheels on curves; friction caused by the wheel flanges on the outside of curves; deflection or undulation of rails; uneven distribution of load on outside or inside rail of curves; and impact imparted to the receiving rail by falling wheels at open or deflected joints.

14-M.—Rail creeping is caused by a wave motion of the rail in the direction of the traffic, brought about by each set of wheels over the rail. The amount of creeping depends on the amplitude of the wave motion thus set up in the rail. This is determined largely by: First, the amount of traffic; second, the weight of traffic; third, the speed of traffic; fourth, the preponderance of traffic in one direction; fifth, the rate of grade; sixth, the elasticity of the roadbed; seventh, creeping is augmented by the application of the brakes, since sliding friction is added.

Question No. 15 and Résumé of Answers.

"Have you had any computations made as to the resultant force causing creeping under specific conditions?"

With a single exception, all the answers to this question have been in the negative; but the following observations have been volunteered:

15-A.—No tests of the longitudinal pull resulting from rail creeping have been made, but in many instances splice-bars are fractured from this cause.

15-B.—Often noticed, especially when trains are traveling down grade, that drivers who are either inexperienced or careless would run too fast, then suddenly check their speed by slamming on the brakes, and then open up and start pulling before the brakes were released. This will cause almost any rail to creep.

15-C.—Have made no computations, but know that, under some conditions, the force may be enough to shear three $\frac{7}{8}$-in. bolts.

15-D.—In 1915, the Engineer of Tests supervised some experiments with the view of determining the pressure required under certain conditions to resist the moving of ties in ballasted track. Prints and descriptive matter showing the device used, together with reports made by the said Engineer of Tests, are attached. On the strength of this test it was concluded that a rail-anchor should show at least 12 000 lb. of resistance before sliding on the rail.

The device referred to in Reply *15-D* consists of a small steel jaw, made of cast steel, $\frac{7}{16}$ in. thick, for attachment to a rail-anchor. The jaw is closed at the operating end except for a saw cut $\frac{1}{16}$ in. wide. To operate it the ends are

sprung apart and a standard copper wire, 0.137 in. in diameter, is inserted. The pressure developed is measured by the amount of crushing of the wire, referring it to a diagram which records the results of a series of tests on such wire. That diagram indicates the following results:

Pressure, in pounds.	Diameter reduction, in thousandths of an inch.
2 000	11
4 000	26
6 000	39
8 000	49
10 000	57
12 000	65

With an apparatus of this kind used at different places and under varying conditions, it should be practicable to determine the pressure between anchor and rail. In a similar manner there might be measured the pressure between rail ends when the track is creeping.

Question No. 16 and Résumé of Answers.

"Is there any information on the subject that you can furnish, not covered by the preceding questions; and, if so, what?"

16-A.—Rails in curved track on which one-way traffic is chiefly pushed ahead of the engine will creep up a heavy grade. On such a stretch where the grade is about 1.9% against the traffic for 2 miles, the creeping is attributed to the grip of the wheel flanges on the ball of the outside rail of curves.

16-B.—The questionnaire does not inquire as to the creeping of track in both directions. Where this occurs, it causes considerably more trouble than movement in one direction only. The movement is not equal, but is generally greater in the direction of the heavier traffic. It is sufficient in the other direction, however, to loosen the rail-anchors unless they are placed on both sides of the ties.

16-C.—Where creeping of rail takes place over a bridge, the rails should be left clear to slide. The practice of slot spiking the angle-bars, or driving spikes at the ends thereof, should be prohibited on bridges.

16-D.—Found from experience that creeping is not so prevalent on those portions of the roadbed which are well drained; and if more time and expense could be devoted to the drainage of muskegs, it would go a long way toward eliminating our present trouble with creeping track.

16-E.—The existing rail-anchors which are on the market can seldom entirely stop creeping, more especially on a single track. This is due to the fact that practically all such devices attempt to provide against creeping in one direction only. It is easier to minimize the evil on a double track; for there the rail-anchors are called on to prevent movement in one direction only. On single track which has been anchored there is, almost invariably, some creeping present. In such cases, therefore, the anchors are placed some in one direction and some in the other. When such a track continues to creep, some of the anchors are backed up; and, if the ballast is partly frozen or even hard, these backed-up anchors are frequently loosened and their usefulness destroyed until re-applied.

16-F.—No trouble could be more serious or difficult to contend with on the track than rail creeping. It is often impossible to do just the right thing; but one thing can always be done, that is, to render "first aid", as it were—and by so doing serious trouble is often averted.

16-G.—Would recommend a foot of sandy loam to make solid bottom, especially on muskeg track, and then a lift of from 1 ft. to 2 ft. of ballast—not too coarse. Would also recommend that the side ditch be put in not closer than 25 ft. from the roadbed, and that a very wide shoulder be used for muskeg track.

16-H.—Bolts and rail anti-creepers should be looked after every day and kept tight, and spikes should be driven to the rail. This, with good, heavy ballast, will check creeping.

16-I.—No method of track construction is known that will eliminate the tendency of rails to creep, except a type by which the rail would be supported under the head instead of under the base, such as the "bull head" rail resting on chairs, as is the practice of some European roads. The only thing that can be done with the type of construction common in the United States is to neutralize the creeping tendency by the application of an equal resisting force in the opposite direction. This can be secured by anchoring the rail to a sufficient number of ties, so that the resistance of the ballast against being displaced by these anchored ties will equalize the tendency of the rail to move forward under rolling loads.

16-J.—On many single-track lines the rails will creep in one direction for a few weeks or a few months and in the opposite direction for the remainder of the year, due to the fact that the direction of maximum tonnage is reversed. An illustration of this would be a "Granger Line" handling grains, stock, and farm products from the farms in the fall, and the remainder of the year carrying supplies to the farms.

The greater the distance between trucks and the greater the distance between axles, the heavier will be the creeping. To illustrate: Very little creeping is found on lines carrying, exclusively, a heavy tonnage of iron ore in especially built ore cars, as compared with lines carrying, exclusively, heavy coal tonnage in the modern type of coal cars. The explanation is that the ore cars are so short and the distance from axle to axle is so small that the combination does not permit of the same height of wave between axles that occurs with long, standard coal cars.

16-K.—The matter of rail creeping has not been given the full consideration that it should receive in track maintenance. It is not only a nuisance and a hazard; but it adds very largely to the actual cost of maintenance of track as well as to the cost of material, through the damage done to the angle-bars and to the rail itself, due to the disturbance of the rail at the joints and the destructive batter on the loosened and displaced connections.

This running of rail has increased to a considerable degree of late years, owing to greater power and heavier loading of cars; and maintenance officers have had to resort to some drastic measures to overcome the difficulty. The only reasonably economical appliance available seemed to be the so-called rail-anchors. There are many different patented devices for that apparatus, some retaining their hold on the rail by friction, and some by attachment to the rail. Generally speaking,

some of these devices were not very effective, some were fair, and a few were very good indeed, but they all need attention from time to time. The trouble is that, after they are applied, the track department does not give them the care they require.

16-L.—Several miles of track were out of operation for more than two years, and no creeping was found thereon during that time.

General Conclusions.

Pending the discussions on this paper, the writer would draw from the preceding evidence the following conclusions:

1.—Rail creeping is a matter of serious import, not only to railroad men, but also to the whole world; for it increases both directly and indirectly the cost of operation and maintenance of every steam railway system, causes great destruction of railroad property and transported goods, and seriously endangers on a large scale the lives of the traveling public.

2.—The principal troubles caused by rail creeping are slewing of joint ties and bunching of all ties; distortion of gauge; loosening of joints; pounding down of rail ends; grinding of rail-heads and wheel flanges; splitting of joint ties; churning and sliding of ties; splitting and breaking of angle-bars; shearing of bolts; cutting of ties and consequent starting of decay therein; loosening, pulling, and shearing of spikes; bunching and squeezing out of ballast; kinking and waving of rails; breakage of track; kicking or buckling of track with consequent derailment; interference with operation of frogs, switch-points, head-blocks, signals, interlocking, crossings, turn-outs, derails, and other track devices; injury to rolling stock due to rough riding track; and, in short, the augmentation of all the difficulties known to trackmen.

3.—Undoubtedly, the primary cause of the fundamental trouble is traffic; because, when a stretch of track is absolutely idle, there is no creeping on it whatsoever. It is true that other influences, such as temperature changes and weather conditions, aggravate the difficulty; but, alone, they are powerless to cause it. Wave action of rails and ties in front of the passing wheels pushes the rails ahead; and, if these are firmly spiked to the ties, the latter move forward, carrying the ballast with them. Again, the traction of the locomotive drivers and the friction of all braked wheels tend to impart a longitudinal motion to the rails. Long cars and widely spaced axles and trucks augment materially the amount of creeping, as does also unbalanced traffic; but these are unfavorable conditions that may very properly be termed unavoidable. The principal avoidable ones, however, are yielding roadbed, lack of thorough drainage, soft or decaying ties, unequal spacing of ties, badly adjusted super-elevation of outer rails on curves, rails too light for the traffic, inefficient or badly maintained rail joints, either scant or excessive provision for rail expansion, inadequate or defectively maintained gauge, poor or insufficient ballast, and improper use of train brakes. If the avoidable evils thus listed were effectively corrected, a large proportion of the trouble experienced from creeping track would no longer exist.

4.—Rail creeping increases with the amount of traffic, although not necessarily in direct ratio; is greater on down grade than on up grade; on a single-track line

is larger in the direction of the preponderance of traffic; in general, is of like amount on tangent and on curve; is often quite different on the opposite rails of a track; is of more serious import on light rails than on heavy ones; is less with firm roadbed than with soft; decreases with improvement in quality of ballast and with augmentation of its depth; varies with changes of temperature, season of year, and weather conditions; is reduced by improving the character of rail-splicing; is not materially affected by the use or non-use of tie-plates; enlarges with the spacing of wheels and trucks; and augments with the speed of trains and with the weights of engines and loaded cars.

5.—In respect to the longitudinal forces in track developed by creeping and the horizontal pull between rail-anchors and rails—this is a question to be settled by experiments and not by mathematics. With all due deference to mathematicians in general and to professors of engineering in particular, it can be stated that the question of rail creeping is not one which will properly lend itself to the manipulation of equations based on hypothetical assumptions, because the governing conditions in respect to wave action of rails and track under traffic are far too varied, complicated, and uncertain to warrant any attempt to investigate the subject mathematically. A few valuable experiments have been made to determine the longitudinal thrust of track from creeping, but the results thereof require confirmation by an elaborate series of tests before being accepted as standard or final by railroad men. The inauguration and carrying out of such a series of experiments is naturally the function of the American Railway Engineering Association, which has already done so much to improve the science and art of railway construction in the United States; and the writer hopes that that organization will soon undertake such a series of tests as will settle this question with sufficient accuracy for all practical purposes.

6.—In respect to rail-anchors or anti-creepers, these may be divided into two general classes, *viz.*, those which attach to the ties and those which are buried in the ballast. The former type is often effective when the track conditions are first class, but usually fail when they are not, and in that case the second type should be used. To be really effective, however, the latter should extend into the ballast well below the elevation of the bottoms of the ties, because otherwise they would move along with the ballast between the ties, which ballast, of necessity, travels with the track. The writer recognizes that, in some cases, the use of such ballast anchors might involve questions of electrolysis of serious import, but believes that these can be satisfactorily solved by proper design, supplemented by experiment. The writer is of the opinion that, in cases of excessive creeping on soft roadbed and ineffectively ballasted track, the best solution of the problem would be the placing of deadmen of either reinforced concrete or creosoted timber in the embankment well below formation level, and connecting both rails thereto by strong, adjustable rods running upward between ties at an angle of about 45° to the vertical. If the creeping tendency is always in one direction, one such rod per anchored rail would suffice; but if it is in both directions, two of them would be necessary. As to how far apart the deadmen should be, that would depend altogether on the tendency of the track to creep. Two or three on each stretch of moving track might first be tried; and if they should prove to be inadequate, more could be added at any time. It is submitted that this method of preventing

creeping ought to be much less expensive than making the track truly first class. The details of the suggested anchorage cannot well be evolved before the maximum rail thrust from creeping is determined.

Instead of the suggested deadman, there could be substituted a pair of fairly long 12 by 12-in. creosoted piles, driven just outside the rails, with their tops at the elevation of the base of the rail, and two 8 by 12 or 10 by 12-in. ties laid flat, spiked or bolted tightly against them, with several pairs of short blocks or spacers between adjacent ties directly under the rails on each side of the pair of piles. Then, if a type of anti-creeper were evolved, which would grip the rail so effectively that it could not slip through, and if several of these anti-creepers were used for each rail and were attached to the ties, a combination would be effected that would absolutely prevent all creeping, unless the thrust were great enough either to snap off the piles or to tip them out of plumb by compressing the earth and moving the ballast near their tops. It is, of course, possible that the thrust would be great enough to effect one of these results; but, if so, the remedy evidently would be to put in more pairs of piles with their adjacent short spacer blocks.

7.—In respect to the prevention of rail creeping on important bridges, the writer's practice is to fasten the rails firmly to the deck at the middle of each span and let them slide on tie-plates out to the ends thereof, where a break in the track over each pier is provided.

8.—Finally, the writer desires to state in relation to rail creeping what he said in print forty years ago in a paper entitled "Railroad Drainage", *viz.*, that the effective drainage of both roadbed and right of way is the main essential in good railroad engineering practice.

In concluding this paper, the writer desires to repeat the hope already expressed near its beginning, that it will be thoroughly discussed not only by the members of the Society, but also by those railroad men of North America who are interested in the subject.

DISCUSSION

ORLANDO METCALF,* ESQ. (by letter).—The author is to be congratulated on the careful preparation of this paper and the logical conclusions drawn from the varied views expressed in reply to his questionnaire. The causes and effects of rail creeping are generally accepted as facts; and, as a rule, both railroad officials and manufacturers of devices to overcome creeping are more interested in the method and means of overcoming it, than in the cause.

The writer feels, however, that one of the most important causes of the creeping of rails has been entirely overlooked, and it may possibly be one of the most serious, as it is not governed by track, traffic, or weather conditions. Any one who has ever ridden over a stretch of new steel, even after an extra gang has given it the usual surfacing, has no doubt felt the frequent jars and bumps of high rail ends at joints. This difference in the height of rail ends, even though a very small fraction of an inch, emphasized as it is under traffic when the wheel load is on the delivery end of the rail, each wheel of the entire train striking the slightly higher receiving end with trip-hammer blows, must be a considerable factor in the forward movement of the rails.

This point is mentioned casually in Paragraph *14-L*, but only as it applies to open or deflected joints, whereas the writer has seen many instances of it on new steel with new joints and very good surfacing.

The writer would also like to see a more detailed discussion of the effect on creeping of using joints without slots for spiking, or even using the slot spikes for mantaining uniform spacing of the ties, but not to retard creeping; also having no anchorage on joint ties to disturb their seat, but preferably to give the rail free movement at joints while anchoring at quarter points.

In anchoring single track, railroad men are too apt to overlook conditions which bring about creeping in one direction with traffic in both directions; and, if Paragraph *14-I* is taken as a basis, it is seen that there is undoubtedly creeping in both directions which is compensated or neutralized by reverse traffic. Therefore, where traffic or grades show creeping predominating in one direction, it must be the resultant difference; and if anchored in this direction only, and ultimately stopped, as it undoubtedly can be, does not the creeping in the opposite direction, although originally less, now predominate? If so, should not anchorage be applied in both directions, although never (as suggested in Paragraph *16-B*) on both sides of the same tie, as this causes constant movement of the tie back and forth, and this churning pushes the ballast back on both sides until there is no resistance to creeping, which, in itself, is the most serious objection to, and strongest argument against, the two-way anchorage secured by any means to both rail and tie.

The writer has heard of experiments made at various times to determine the resultant force of creeping; but, from careful observation over a period of years and under widely different conditions, he doubts very much, even if it should be accurately determined, that it would prove of much material value in solving the problem, nor could it be used generally as a basis of the anchorage required.

All the causes which affect creeping vary too greatly and no two can be accurately fixed in relative importance, therefore it would require more than a mathe-

* Vice-President, Anchor Company, Milwaukee, Wis.

matical genius to make even a comparative estimate under any given condition; and every installation would have to be computed on the local conditions governing it, whereas even now too little attention is given to local conditions by those responsible for the results.

The greatest trouble to be met and overcome is misuse and even abuse of various types of rail-anchors. The writer does not mean to intimate that this is deliberate, but rather through force of circumstances, due to the necessity of endeavoring to anchor 50 miles of track with only enough anchors furnished properly to anchor 5 miles, and in many cases the results are so poor that casual inspection makes one skeptical in regard to all types of rail-anchors. Too frequently only the worst conditions are seen, and partly provided for, and no thought is given to the origin of the trouble, possibly several miles back, and the device used so sparingly and so futilely fails to meet the needs, since it was really used in the wrong place.

More intelligent study of local conditions to be overcome, and full co-operation on the part of all concerned properly to provide for them, would soon bring results so satisfactory that few would question the means to be used to meet any given condition. The economy of proper use—liberal though not extravagant—of many types of rail-anchors now available, would thus be soon recognized. The writer cannot agree with the recommendation under "General Conclusions," not that it would be ineffective, but in that the cost would be so much greater and the results probably little, if any, better than the proper application of various types of rail-anchors now being used.

C. A. Disbrow,* M. Am. Soc. C. E. (by letter).—Railroad track is a subject of great interest. It should appeal especially to those engineers who, at times, enjoy setting theory aside and making use of experience and "horse sense".

In general, the writer is in accord with the conclusions with which Dr. Waddell closes his paper, particularly that in Paragraph 8, where he says that "the effective drainage of both roadbed and right of way is the main essential to good railroad engineering practice."

The effect of water in the roadbed is very clearly demonstrated in countries where there are only two seasons, the dry and the rainy season. Track which is in fairly good condition during the dry season, may become almost impassable during the rainy season, because of the water in the roadbed and right of way. The writer has seen track overgrown with grass and weeds and needing much done to line and surface, yet this work was left undone and all efforts were directed toward drainage, because of the approaching rainy season. If ample drainage was not provided, water in the roadbed would render the track practically unsafe for traffic before the season was over.

Although drainage is essential in order to have good track, it is not sufficient in itself to prevent creeping, under all conditions, and the creeping of track is a source of much maintenance trouble and expense. The rail-anchor or anti-creeper, now in general use, has done much to overcome this trouble. Before separate rail-anchors were used, rails were first held against creeping by making anti-creepers of the splice-bars. The writer has been told that the angle-bar splice was designed to provide a foot member which could be slotted to receive the track spikes and thus hold the rail against longitudinal movement.

* New York City.

This practice of making the splice-bar act as an anti-creeper should now be discontinued, and some railroads have done so. In the writer's judgment, the splice-bars have done their full duty when they properly support the rail ends, and by properly supporting, is meant uniting the two rail ends so as to make the rail at the joints as nearly like the rail itself as possible. In order to unite the rail ends, it is necessary to add metal in the form of splice-bars, thereby creating a condition that does not exist elsewhere along the rail. If these splice-bars are then spiked through slots, still another condition is added which does not exist elsewhere along the rail. This is a condition that can, and should, be avoided.

The point the writer wishes to make is that the joint should not present a mass of metal fastened so as to be a buffer to the approaching wheel-loads. It should be as nearly as possible like the rail in flexibility, thus allowing the wave motion to pass on uninterruptedly. The rail at the joint should be spiked as the rail between joints is spiked. If the rail creeps, add sufficient anti-creepers to prevent creeping, but do not make an anti-creeper of the rail joint.

C. D. PURDON,* M. AM. SOC. C. E. (by letter).—The writer recollects, indistinctly, a statement from a correspondent, published many years ago in some technical periodical, to the effect that in a railway track running north and south, the east rail would creep north and the west rail south, and inquiring as to the reason. The writer does not recollect further correspondence, but about the same time some discussion arose on the creeping of track on the Eads Bridge at St. Louis, Mo. As well as he recalls, the statement was that the tracks crept about 10 in. per day in the direction of the traffic (the bridge being double-tracked) and after all attempts at anchoring had failed, expansion devices were used and the rails allowed to creep. In the opinion of the late C. Shaler Smith, M. Am. Soc. C. E., the cause was "wave motion".

In 1904, a paper† on this subject was presented before the Society by Samuel T. Wagner, M. Am. Soc. C. E., in the discussion of which G. Lindenthal, M. Am. Soc. C. E., stated that, in his opinion, the cause was the dynamic blow from moving wheels, caused especially by the shortness of the rails.

Dr. Waddell refers to the replies to his questions as "votes", but replies to questions on such a subject can hardly be called a ballot. Most of the answers are in favor of some kind of rail-anchor, but they refer generally to track on embankments; and Dr. Waddell is in favor of forcible measures. This means would be open to criticism, as the resultant force must be considerable, judging from answers to Question 15, "splice-bars are fractured from this cause", "force may be enough to shear three $\frac{7}{8}$-in. bolts". If two or three of the "strong adjustable rods" should get loose or break, they would be very likely to cause disaster.

Little is said, however, as to anchoring track on bridges or trestles. The writer is of the opinion that this should not be done except, perhaps, as mentioned by the author in Paragraph 7 of his "General Conclusions"; spiking in the slots will split the ties, and the projecting flanges of the angle-bars will also split the ties when they encounter a spike. The flanges of the angle-bars on the Thebes Bridge were sheared off to prevent this, and, indeed, the old-fashioned fish-plate would be

* St. Louis, Mo.

† "Some Notes on the Creeping of Rails," *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 466.

preferable on a bridge to any kind of angle-bar, because there is no deflection at the joints, as an embankment requires no more than will keep the rails in line.

L. C. Hartley, Chief Engineer of the Southern Illinois and Missouri Bridge Company, which owns the Thebes Bridge, has stated that from September 1st, 1917, to July 1st, 1918, the west rail on the south-bound track on this bridge, ran 123 ft., which is equal to a rate of 12.3 ft. per month; from July 1st, 1918 to December 1st, 1920, the same rail ran 495 ft., at a rate of 17.7 ft. per month.

The writer submits a rough sketch (Fig. 1) of a trestle 9 327 ft. long, on which creeping rails caused a great deal of trouble before expansion joints were used. It would appear to him that much of the movement was caused by trains stopping for the draw-span, on the 5° curve, as the outside rail crept north, while the inside rail crept very little, and that little south; but before expansion joints were used, constant work was required all along the trestle by reason of the joints being too tight.

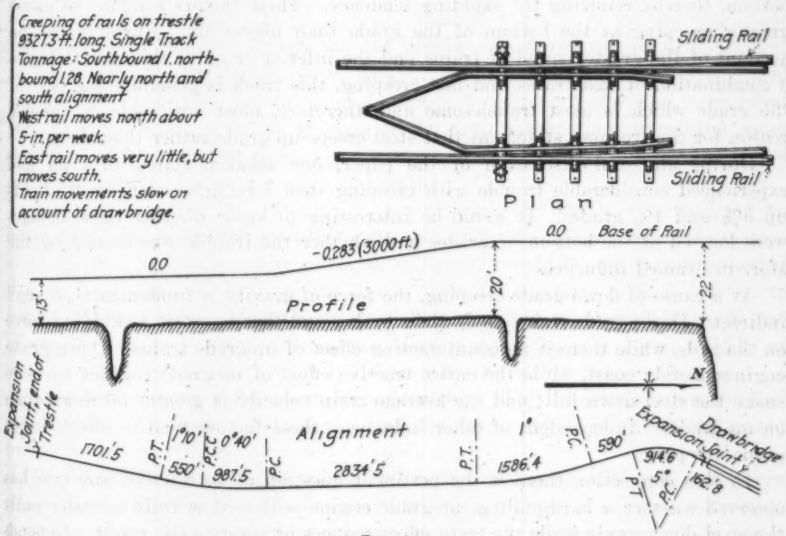

FIG. 1.

LEONARD C. JORDAN,* ASSOC. M. AM. SOC. C. E. (by letter).—The author is to be commended for his "cards down" policy in the presentation of this paper for discussion. His request for comments and further information will result in more good to the Profession than an attempt to draw final conclusions from the data already in hand.

Dr. Waddell has brought up a very important matter for discussion. Those who endeavor to belittle the importance of the subject should be reminded that they are merely calling attention to their own lack of contact with this extremely perplexing problem. Although it is true that rail creeping causes very little trouble under certain conditions of heavy track and light traffic, there are innumerable cases where the combating of this difficulty is a job for a Job.

* New Rochelle, N. Y.

The writer can recall some cases of decidedly persistent steel creeping experienced on the Union Pacific Railroad in Colorado and Wyoming where he spent his summer vacations during school days and also the year following his graduation, while acting as foreman of floating gangs on steel laying, general track repairs, etc. In one instance, he surfaced a stretch of dirt ballast track on a heavy grade where the bulk of traffic was down grade. In ten days the joint ties had been moved fully 10 in., or entirely off their solid beds and into the soft dirt beyond. On the opposite side of the hill the steel crept up a grade of approximately 1½% in the direction of heavy loads.

One point which seems to have been overlooked in the questions and answers is the fact that when steel creeps up grade on a single-track line, due to unbalanced traffic, this tendency is far greater at the lower end of the grade than elsewhere. Up-grade trains have a greater velocity at this point than farther up, due to running for the hill, and down-grade trains release brakes before reaching the bottom, thereby relieving the skidding tendency. These factors combine to cause greater creeping at the bottom of the grade than higher up. Furthermore, on account of the greater speed of trains and the inferior track at this point, due to a combination of fast trains and fast creeping, this track is generally the part of the grade which is most troublesome and, therefore, most noticeable, a possible reason for the frequent statement that steel creeps up grade rather than down.

During the oral discussion of the paper, one speaker stated that he had experienced considerable trouble with creeping steel forcing wooden trestle bents up 3% and 4% grades. It would be interesting to know whether these bridges were located at the bottom of grades and whether the trouble was caused by the afore-mentioned influences.

As a cause of down-grade creeping, the force of gravity is fundamental, though indirect. Down-grade trains apply their brakes and thereby exert a skidding force on the rails, while there is no counteracting effect of up-grade trains. Down-grade engines merely coast, while the entire tractive effort of up-grade engines tends to snake the steel down hill; and the average train velocity is greater on down than on up grades. Independent of other influences, these factors tend to affect down-grade creeping.

In this connection there is the pertinent question as to whether any one has observed whether a hard-pulling, up-grade engine with a slow train actually pulls the steel down grade while the train effect reduces or reverses the result. Dependable data on this question might be enlightening.

Throughout the paper, and especially in the answers to Question 9, the fact is brought out that creeping is greater where there is great amplitude in the wave motion due to light rails or springy roadbed or both. Swampy sub-grade is justly blamed for causing the creeping. This wave motion, under cars and in front of engines, is merely the deviation from a straight line in the rails caused by depressions under the wheels. The depressions are deeper for swampy ground, springy ballast, weak ties, light rails, and heavy loads; therefore, all these factors tend to increase creeping.

The writer frequently has lain down very close to the track in order to sight along the rail while a slow train was passing. The appearance of these waves, as they move along with a high point under each car center, would seem sufficient

to account for a large part of the creeping. The length of the rail in a wave between trucks is greater than the straight-line distance between the same points. The difference between these lengths measures the actual creep per car due to wave action because, as it advances, each truck rolls all points of the rail down to a straight line. When a car is so long or the track so flexible that the top of the wave assumes the normal unloaded position, then it would be difficult to imagine that a greater car length would further increase creeping. Creeping, however, will be reduced by car lengths so short that the wave amplitude is reduced. Probably the impact due to high train velocity causes deeper depressions under the trucks, or greater amplitude of the wave, and, therefore, greater creep. Yet we must not lose sight of the fact that there are other causes of rail creeping, and that wave motion is merely one influence.

As a mathematical illustration of wave effect only, if a wave is composed entirely of circular arcs of equal radii and the length is 20 ft. between wheel centers and the height is $2\frac{1}{2}$ in., the hypothetical creep is $\frac{1}{8}$ in. per car. Less amplitude and a sharper curving near the wheels, a more probable condition, might easily cause a greater creep. Although Dr. Waddell has very properly excluded mathematics from this discussion, it is proper to point out that the wave effect per car can be only a small fraction of an inch rather than something more alarming. Yet from this cause alone a wave effect of only $\frac{1}{100}$ in. per car would result in a creep of $\frac{1}{2}$ in. for each long train.

Answer *8-L* states that creeping increased when light rails were replaced by heavy ones. In connection with this statement, there should have been information as to whether the track was re-surfaced, as usually would be the case, or whether the new steel was laid down on the ties in their old positions, with the resulting impossibility of slot-spiking the joints. The only fair comparison between the effects of light and heavy rails in this respect would be with equal solidity of roadbed, equal slot-spiking, equal traffic, and the same weather conditions.

Answer *13-G* advises the use of 12-ft. joint ties. This would require costly and objectionably wide ballast shoulders. Also, the ties would likely have to be moved to new locations with each renewal of steel.

Dr. Waddell suggests an occasional substantial anchor by driving piles between ties or by planting a "deadman" under the track. There is a serious objection to such a plan. There would be an opening of joints beyond each anchor and a buckling tendency just back of it. The severe strain thrown into one rail joint very probably would cause a dangerous break at a critical time. Such anchors would have to be at frequent intervals to be effective, which would be costly. Also, the proposed spacer blocks between ties would be loosened by each slight reversal of creep.

It is time to point out the strange fallacy of laying rails with broken joints. Joints are sure to get low at times, and when they are staggered, they give cars a rocking motion which tends to throw the rails out of line, while even joints merely cause a vertical vibration. Also, with even joints a relatively better support can be provided by choosing the largest ties for the joints during renewals. With even joints there will be no slewing of ties by creeping steel, such as is cited repeatedly in the paper. Another advantage will be mentioned subsequently.

Old, experienced track foremen argue that for these reasons the change to staggered joints from the earlier practice of even joints was an unpardonable mistake. It is sometimes contended that the presence of at least one rail continuous past any point of track provides lateral rigidity. The absurdity of this notion will be recognized readily when thought is given to the extremely small lateral strength of any standard rail. No considerable rigidity can be given to the track by a rail so flimsy that an ordinary man can deflect it into a sharp curve by the strength of only one hand, as is done in lifting an end of a rail when lying on its side.

There seems to be a general agreement that creep will be eliminated when there is a rigid connection with several ties per rail. The usual forms of anti-creepers are effective in only one direction, and they are loosened when even a small amount of creep occasionally takes place in the reverse direction. The solution then would seem to lie in a rigid attachment gained by bolting splice-bars to the rail at the center and by slot-spiking to the ties. For this purpose some foremen use the halves of broken splices. When even joints are used there is no slewing of joint ties. Such intermediate splice-bars also act as rail braces and thus give additional strength to the track. If this practice should prove to be generally adopted, it would be well to roll a light section splice-bar which would serve this purpose as well as the standard bars, and which would be considerably cheaper than a bar heavy enough to form the splice.

In the writer's opinion there is ample opportunity to put an end to ordinary creeping by a different arrangement of the joints. By using even joints and special tie-plates extending over two or three ties at each side of the joint, all tendency to creep would be communicated to several ties, as many ties as would be found necessary. These tie-plates should be merely long lengths of the plates from which tie-plates of the shoulder type are sheared. They would have holes for spiking to all ties just like any tie-plates. The rail anchorage would be formed by heavy spikes, preferably screw spikes, into the joint ties and through the splice-bar slots. There would be no slewing of ties, and the arrangement would be equally effective in the two directions. The cost of such tie-plates would be small, as there would be no field drilling and no other special work. The plate would replace nearly half its length of shorter plates of the same section, a plate extending over six ties would weigh only about 50 lb. more than the small plates replaced. This plan surely is worthy of a trial before being rejected as inferior to some device more costly and less certain of producing the desired results. The addition of a clamp between the joint ties and holding the plate to the splice-bar flanges would strengthen the joint to some extent.

It is to be hoped that Dr. Waddell's paper and its discussion will result in an inexpensive method of aiding track to carry its ever-increasing burden.

SAMUEL T. WAGNER,* M. AM. SOC. C. E. (by letter).—On September 7th, 1904, the writer presented a paper to the Society entitled "Some Notes on the Creeping of Rails"† containing observations which were made by request of Dr. P. H. Dudley, who was at that time Reporter to the Fifth International Railway Congress and who desired special information under the heading, "Methods for Pre-

* Philadelphia, Pa.

† *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 466.

venting Creeping, Especially on Double Track Lines and on Steep Gradients". As the Philadelphia and Reading Railway Company had no data to answer this question satisfactorily, the writer was directed by the late William Hunter, M. Am. Soc. C. E., then Chief Engineer, and the late Theodore Voorhees, M. Am. Soc. C. E., then Vice-President, to make a number of actual measurements under varying conditions of grade, speed, alignment, and character of roadbed, to endeavor to determine a proper answer. The observations, although carried on for more than a year, did not develop anything of definite value. It was interesting to note in the discussion that no other road, with one exception, had any definite data to give from actual measurements.

On reading Dr. Waddell's paper, the writer was at first impressed with the fact that no more actual measured data had been added to the literature on the subject, and was disappointed. On second thought, it has appeared that, after all, probably any elaborate records of such creeping would prove of no value commensurate with the cost involved in making them, on account of the great number of conditions controlling the creeping and the greater number of varying conditions as to character of roadbed, drainage, etc., which would influence the results. It is extremely probable, therefore, that the method used by Dr. Waddell in attacking the solution of the question is the best. The result will be a composite one and not based on actual measurements, but, as such, will be of value.

The writer submits the following itemized discussion of Dr. Waddell's "General Conclusions:"

1.—He fully agrees with the conclusion reached under this heading, *viz.*, the importance of the subject.

2.—He also agrees that the author has undoubtedly covered all the principal difficulties caused by creeping.

3.—It is believed that the action of changes in temperature is an almost negligible factor by itself. Yielding roadbed, lack of thorough drainage, light rails, and other conditions tending to allow wave motion in the track, are believed to be the most potent factors, in spite of the opinions advanced by some engineers. Traffic, and especially heavy traffic, together with the previously mentioned factors, is believed to be of the greatest importance. The experiments which the writer made in 1903 clearly show this, if nothing else.

4.—He agrees generally with the conclusions reached by the author. It is to be noted, however, that even on very heavy grades, the creeping is up grade if the heavy traffic is in that direction.

5.—He agrees with the author in the conclusions reached, with the exception, as already stated, that he very much doubts the advisability of any elaborate set of experiments being made, on account of the practical impossibility of covering all the varying conditions which occur in the condition of the track.

6.—This item (rail-anchors or anti-creepers), of course, covers the real crux of the question, *viz.* how can the creeping be reduced or eliminated? There is no doubt that, in many cases, rail-anchors are a great help, but the use of "deadmen" placed below the ballast is questioned as a general cure. It may be satisfactory in certain special cases, but the idea of digging down and disturbing the roadbed does not appeal to the writer as a good solution. The less the roadbed below the

ballast is disturbed, the better, and the placing of any metal in such location does not seem to be advisable on account of the danger of corrosion and the great difficulty of replacing it without again disturbing the track. Although such a method, as stated by the author, may cost less than keeping the track in first-class condition, the money spent on the track to keep it in that condition, is always money well spent, and it is certain that, although the creeping may not be entirely eliminated by such expenditure, it at least will be reduced, possibly to a point where it may not be objectionable. The writer fears that with the use of the contrivances suggested the creeping might continue if the maintenance of the track were not kept up to the proper standard.

8.—The writer fully agrees with the author in his statement with reference to drainage, but would also add to it that proper maintenance of the track in its more important details, as to character of track, maintenance of joints, weight of rail, etc., is of equal importance.

What shall be done to prevent this creeping is the real question, and is a most difficult one to solve. It is hoped that the discussion of the paper will bring forth from those who are directly responsible for track maintenance, practical advice as to the best solution of the problem, which it is believed can only be solved in this way.

F. W. GREEN,* M. AM. SOC. C. E. (by letter).—The compilation by Dr. Waddell of opinions on the subject of creeping rails is interesting as well as timely. The subject is one which has given officers charged with the responsibility for the maintenance of railway track a great deal of concern. Numerous expedients have been tried, all of which have been only partly successful.

Although the causes of creeping rails may be more or less obscure and possibly various, the writer has reached the conclusion, in his own mind, that rails creep because of the resistance to traction of cars pulled by a locomotive. To make the principle clear, imagine the wheels of the cars locked by brakes being set, on level tangent track, and with a static coefficient of friction of 0.34. For each ton of weight on the wheels, the force necessary to move the cars would just equal 0.34 ton. After movement started, of course, the coefficient of friction would become dynamic, instead of static, and would decrease inversely with the speed. With a tractive force of 0.34 ton, the rail could (*a*) remain perfectly stationary, and the car slide on the rail, or (*b*) the car could slide and the rail slide, or (*c*) the car could remain stationary and the rail slide. In any event, the force would be exercised along the line of least resistance.

With the brakes released, assume that the cars are loaded so as to produce, say, 5 lb. per ton tractive resistance. The tractive force necessary to move the cars now becomes $\frac{5}{2\,000}$ or 0.0025 ton. Here, the same three phenomena, as mentioned previously, may occur: (*a*) car move with the rail stationary; (*b*) car and rail move; or (*c*) rail move with the car stationary, relative to the rail.

It will be observed at once that the tendency of the rails to move increases as the traction resistance increases, from which it follows that a train moving with considerable weight and speed, with air-brakes applied, will probably increase the tendency of the rails to creep.

* St. Louis, Mo.

In the writer's opinion, however, the tendency of rails to creep is greatest in the vicinity of curves, except possibly on long, descending grades, where the speed of the train has to be controlled by air-brakes. It is customary on all railroads to super-elevate the outer rail on curves for passenger train speeds. Any train passing over the curve at the precise speed for which it is elevated will have equal wheel reactions as between the inside and outside rails. Freight trains ordinarily traverse curves at slower speeds, and as the speed decreases, the wheel reaction becomes less than one-half the weight of the truck for the outside wheels, and correspondingly more than one-half the weight of the truck on the inside wheels. By reason of the length of arc on the inside rail being shorter than the arc on the outside rail for equal angles subtended at the center of the curve, there is a tendency for the inside wheels to run ahead of the outside wheels. In other words, the axle joining the wheels will not be radial. The inside wheels will continue to run ahead until the flange pressure on the forward outside wheel and the rear inside wheel of the truck reaches a point in excess of the weight carried on the wheels, multiplied by the coefficient of friction. At this point the truck must rotate in an approximately horizontal plane, and one of two things must happen: (*a*) the outside wheel will slide forward, or (*b*) the inside wheel will slide backward.

It is believed that when freight trains traverse curves at speeds less than those for which the curves have been super-elevated, the outside wheels carrying the lesser load will slide forward. When this occurs, the tendency of the outside rails to slide in the direction of the train movement is augmented materially. Incidentally, when this takes place, it is believed that the rolling effect on the inside rail occurs, which causes the top of the inside rail to flatten out and produces what is commonly called "fins". It is thought that this explains why rail will run at certain parts of the line in one direction on one side to a greater extent than on the other.

The writer has found that a very effective way of reducing the creeping of rails is to maintain the track gauge at 4 ft. 8½ in. on curves. As the gauge is permitted to widen, the trucks make longer chords before slewing, and when slewing occurs, it is with some violence, not only increasing the rail wear, but also increasing the creeping.

It would be interesting to know whether the tendency which the writer has observed, and from which observations he reaches the foregoing conclusions as to the cause of creeping rail, has been observed, with similar results, by others. It has also been the writer's experience that the tendency to creep is much less marked where the rail has been laid and maintained with proper expansion. Where maladjustments of expansion occur, the tendency for rail to creep is especially marked, and the difficulty of anchoring it successfully against creeping greatly augmented.

PAUL HAMILTON,* ESQ. (by letter).—The serious troubles caused by rail creeping on railroads are fully brought out in the replies to Dr. Waddell's questionnaire, and there seems to be no need of elaborating on them. The conditions which bring about rail creeping are well known to railroad men engaged in track maintenance. Creeping is generally caused by the wave motion set up in the track by the passage

* Engineer, Track and Roadway, Cleveland, Cincinnati, Chicago & St. Louis Railway, Cincinnati, Ohio.

of trains, the direction of the creeping coinciding with that in which the train is moving at that particular time.

If the roadbed is unstable, such as occurs over swamps and bogs, the wave motion is greatly augmented and the creeping is correspondingly increased, so that the track is dragged back and forth with the passage of each train. On a stable roadbed, rail creeping occurs when the preponderance of loaded car traffic is in one direction, the direction of creeping coinciding with that of the heavy traffic.

The tendency to creep under traffic is aggravated where the rail is of light section, drainage is poor, ballast is of poor quality and insufficient section, or the ties are decayed and improperly spaced.

It has been the writer's observation that creeping occurs on down and up grades to about the same extent, except in cases where it is due to unstable roadbed. Curvature has little if any effect on the amount of creeping.

The prevention of creeping is best accomplished by reducing to a minimum the sources of the wave action in the track which cause it. The steps to be taken in any particular case will depend on conditions.

The writer is familiar with a single track constructed over a stretch of boggy ground. The fill was 4 to 5 ft. in height. Under passing trains the track would creep from 6 in. to 1 ft. The wave action set up in the roadbed by passing trains could be distinctly felt in the ground adjacent to the roadbed for a distance of 100 ft. The track over this bog was raised from 6 to 8 ft. on locomotive ashes, and widened to 24 ft., the weight of rail—65 lb.—was increased to 80 lb. per yd., and double shoulder tie-plates were applied to each tie. Finally, the track was ballasted with washed gravel ballast. The result has been that creeping has been eliminated, and it is no longer a source of trouble. It is the writer's opinion that most cases of creeping over swamps and bogs can be cured, for all practical purposes, by similar methods of treatment.

Where creeping occurs on a roadbed that is stable, of sufficient width, and well drained, the weight of rail should be increased to not less than 80 lb. per yd. (preferably 90 lb.); the track should be fully ballasted with a good quality of ballast, and well drained, a good condition of ties should be maintained, and rail-anchors should be applied. When severe creeping occurs under heavy traffic in one direction, it is a common mistake to apply an insufficient number of rail-anchors. Under such circumstances it may be necessary to apply not less than twelve anchors to each panel of track. It is essential to the proper performance of any rail-anchor attached to the base of the rail that the ballast be kept away from it. There are several types of anchors on the market that will give satisfactory results, if properly maintained, and if a sufficient number are applied to the track.

It is the writer's opinion that creeping of rails can be reduced to a minimum, so as to cause little trouble, if the track is given the treatment just outlined. In the event that the means cannot be secured whereby the physical condition of the roadbed and track structure can be improved, resort must be had to attempts to secure some relief from the effects of creeping, and to minimize its action. One means of accomplishing this is by the installation, at critical locations, of switch-points placed in the track in trailing position for the direction of greatest creeping. Such installations prevent the cumulative effect of creeping by breaking the continuity of the rails. They should be used only in extreme cases.

The writer knows of no successful attempts to anchor the track to the roadbed, and doubts the practicability of doing so. For such means to be effective, it would be necessary to place them at close intervals, and the expense of installation and maintenance of any such system would probably approximate the cost of improving the track structure so that it could be held with rail-anchors.

In conclusion, it may be said that the phenomenon of the creeping of rails is not mysterious and can be traced to well known causes, and that in the majority of cases it is entirely practicable to prevent it by methods of treatment now used.

G. D. SNYDER,* M. AM. SOC. C. E.—The speaker considers this paper to be of great interest and value and will discuss briefly the subject in general.

A well known writer, who lived before the age of railways, remarked on the lack of wisdom with which the world was governed and the same is true of the engineering world to-day. Specialization has been carried so far that some super-engineer is needed to take charge and remind us that we are working for a common end. For instance, a railway may be classified into two general elements—the track and supporting structures being the static elements, and the vehicles that run on the track, or the dynamic elements.

The engineers who have to do with these two essential elements of a railway have developed along such independent lines that it sometimes seems that they forget that each is dependent on the other, that the track is useless without the motive power and vehicles, and that the latter are useless without the track. Of course, they have had to agree in some things—the track gauge, for instance, and loading on bridges. For although most bridges are designed for a train loading in excess of any actual loads in existence at the time of their construction, they must be renewed or strengthened in a relatively short time, not because the structures have deteriorated and are unable to support the loads for which they were originally designed, but because the train loads are far in excess of those for which the bridge was designed. This is also true in reference to the track, the difference being that the track, having a more continuous support, is not so likely to collapse when overstressed, as a bridge.

The motive-power engineer has been constantly increasing the weight of his vehicles since the inception of railways, and the roadway engineer has been trying to keep up with him. These heavier loads have been largely responsible for the reduction in the cost of railway transportation that prevailed from the early days up to the beginning of the World War, so that within limits it was a good thing, and the speaker's only criticism is the lack of co-operation between these two classes of railway engineers, and the lack of vision displayed by both as to the future.

There is an ocular demonstration of this development in the Grand Central Terminal, New York City, where the train and track, model of 1831, are on exhibition. The equipment in use to-day can also be seen, and one gets an impression through the eye that is not conveyed by mere dimensions. One is first impressed by the stack that looks to be very tall, yet its height above the top of the rail is about the same as that of a locomotive of to-day. Owing to the increase in size of the boiler, the stack has almost disappeared. The boiler has so increased in

* New York City.

height that in many cases there is no longer space between its top and the clearance line for the headlight in its accustomed place and, therefore, it is placed in front.

This is merely an indication of increase in dimensions and consequent weight, and one wonders how long this will continue in the future. Owing to the existing limitations as to clearance it would appear that the loads could not continue to increase at the same rate as in the past, unless the limiting structures are rebuilt, as the dimensions of coal cars and locomotives are approaching these limiting lines.

One phase of this subject has not been referred to, and that is the effect of electric traction on rail creeping. It will probably be found that it is not as great as with steam locomotives on account of the more uniform torque, as compared with a reciprocating engine. The amount of creep is reduced, but it is not eliminated. There are several hundred miles of underground railway tracks in Greater New York which are of interest because the conditions differ so materially from those that prevail on a trunk-line railway. The temperature conditions, for instance, are nearly uniform throughout the year. Some of these tracks are laid with ties embedded in ballast and some on wooden blocks set in concrete. Screw spikes are used in some portions and driven spikes in others. Although the conditions are so different, these rails creep in a manner similar to those outside but, perhaps, to a lesser extent. The contact rails also creep to some extent, and in some cases these have been heavily anchored. Multiple-unit traction is used in all cases except for the trains entering the Pennsylvania and New York Central Terminals. The frequency of trains is high, up to 40 trains per hour, as many as 300 to 350 cars passing in 1 hour.

Two very interesting papers on this subject have been discussed by the Institution of Civil Engineers,* in which an experiment is described. Laths, 1½ by ⅜ in. by 2.5 meters, were used to represent rails, which were placed on an ordinary work-bench, and wheels 12 in. diameter of varying weight were lightly rolled by hand along them, and the laths were found to move in a manner similar to rails in track.

It would seem that a point has been reached where the whole subject of track design and construction should be restudied, not only by the men who design and maintain the track and roadway, but in co-operation with the men who design and operate the equipment that moves over the track and roadway. This Society is broad enough to comprise all branches of engineers, and it would seem therefore to be the proper body to institute such a study. It would appear better not to limit such an investigation to the membership of the Society, but to form a committee of the ablest men on this subject, whether members or not. Such a committee should not only investigate rail creep, but also the increasing loads on roadbed and structures.

There occurred, at a recent meeting, a discussion on the question of the relative advantages of steam and electric traction on railways.† This meeting was largely attended by mechanical and electrical engineers, but roadway engineers did not take part in the discussion, although the track is a vital element in this problem.

Similar conditions prevail in highway engineering. Prior to the advent of the automobile and motor truck, practically stable conditions of practice had been

* "Rail-Creep", by Frank Reeves, *Minutes of Proceedings*, Inst. C. E., Vol. CCV (1918), p. 227, and "Creep of Rails", by Harry Powell Miles, p. 243.

† "The Electric Locomotive, Pro and Con.", Railroad Section, Am. Soc. M. E., and New York Section, Am. Inst. E. E., in Joint Meeting, October 22d, 1920.

reached. The density of traffic increased up to the maximum possible, but the weight and speed of the horse could not be increased very fast by breeding. With the advent of mechanical means of transport the whole art of road construction has had to be restudied and revised to meet these new and unprecedented conditions.

D. L. TURNER,* M. AM. SOC. C. E.—The fact that considerable rail creeping has occurred in the subways, has been referred to by Mr. Snyder. That is true, but the companies have not arrived at any standard of treatment. As a matter of fact, originally a certain amount of anti-creeper protection was specified, but neither one of the operators thought it wise to place that protection at once, preferring to let experience determine the amount of protection that would be required, and its use. At present, as a result of their experience in operation, both operators are gradually installing creepers of the type provided for in the contracts.

In time, the result of this experience and experimenting will undoubtedly provide some valuable information as to the protection that is required on subway or other rapid transit track.

DANIEL BONTECOU,† M. AM. SOC. C. E.—The speaker is glad to testify to the very thorough way in which the causes and effects of rail creeping are stated in this paper. The evidence on which the author's conclusions are based is evidently given by practical trackmen and the conclusions drawn should in the main be generally accepted by railroad engineers. Although the replies to the various questions conform to long established opinions and practice, a real service is rendered by formulating them for the benefit of those who from time to time need assistance in details of track maintenance.

The creeping of rails appears to be due in a minor degree to gravity on grades, to temperature changes, and, principally, to wave action caused by traffic. The creeping due to gravity and temperature can certainly be prevented by properly spiking rails to ties, and the speaker believes that the creeping due to wave action can be so far minimized as to become of little economic importance if the rails are well and sufficiently spiked to ties in ballasted track. Wave action in track consists of the deflection of the rails between ties and the general wave motion of the elastic track between wheels.

It is difficult to see why the rail deflection between ties should contribute to creeping, but there may be an effect similar to that from deflection of a vibrating bridge span which causes the free end to travel back and forth on the rollers in time with the vibrations. In this connection the speaker recalls a statement that soon after the completion of the Eads Bridge at St. Louis, Mo., there was an extraordinary amount of creeping of the rails on the structure—about 12 in. each day—and that the trouble was corrected by providing a more continuous support under the rails. There can be little doubt but that the wave action of the entire track structure is the principal cause of creeping, the wave being ironed out by the advancing train much as a road roller moves loose stone ahead in constructing a macadam road.

The problem is to make the track so elastic as to afford smooth riding quality and at the same time prevent vertical motion. The answers to the author's ques-

* New York City.
† Mamaroneck, N. Y.

tionnaire are very definite in claiming that rail creeping is much reduced or altogether prevented when there is a substantial well maintained track on a drained foundation. The speaker's practice has been to depend on a stiff rail, well tamped rock ballast, and the simple expedient of spiking the rails to the ties as often as necessary, using old slotted splice-bars for a connection.

The remedy proposed by the author for rail creeping on track that is not first class, that is, using piles or "deadmen" to anchor the rails, will probably not appeal to maintenance-of-way men, except as a last resort, and it is very difficult to see how the anchors can be satisfactorily attached to the rails.

For many reasons, good track is a prime necessity of successful railroad operation, and its economic advantage is far greater in many respects than in simply reducing rail creeping, so that in place of spending money in attempts to anchor the track to the subgrade, it would be better to use it in track improvement.

In general the author's "Conclusions" seem to lay rather too much emphasis on the risk and cost due to rail creeping. In the speaker's experience in maintaining 1 400 miles of track, with rail weights of from 56 to 80 lb. and traffic up to 100 trains per day on single track, he has considered that rail creeping was a nuisance and required watchfulness and intelligent care, but was not a serious menace to safety of trains, and did not involve serious expense.

The supreme importance of good track is more generally recognized now than formerly, but there are many cases like that of an important line on which there was an expert report that the equipment was twenty years ahead of the track. Therefore, the speaker deprecates any suggestion from an engineering quarter of expense on track that does not directly improve the track itself, irrespective of the creeping of rails.

B. H. BRYANT,* M. AM. SOC. C. E.—It has occurred to the speaker that a brief outline of his own experience dealing with the particular evil treated of in Dr. Waddell's paper, may be helpful to those who are seeking an economical and effective method of overcoming it.

The speaker has spent twenty-nine years on various mountain railways between British Columbia and Texas, and throughout that time has struggled with the numerous troubles that result from rail creeping. He was unable to determine the cause of the phenomenon, as it seemed to result from different influences on different portions of the different roads, but Dr. Waddell has mentioned many of the influences to which expert trackmen throughout the country have attributed this troublesome development.

One of the railroads with which the speaker was connected had 160 miles of 2%, 3%, and 4% grades, and it was on these heavy grades that the creeping of the rails was most pronounced, and much to his surprise it was not always down grade.

Because of the steepness of the grades, the changes of elevation were rapid and very marked, the range of elevation being from 5 000 ft. above sea level to about 11 600 ft., and down to 4 000 ft. As a result a wide variation of climate was encountered on a trip over one passenger division, a distance of about 400 miles, particularly during the winter months.

* New York City.

About 100 miles of the line was above 9 000 ft., the winters were long, and, on some sections, the snow was deep, as the annual snowfall was from 40 to 45 ft., and there were seasons when the banks of snow along the track were 10 or 12 ft. deep in June. The frost might enter the ground in November and leave it in July, making the season during which useful track work could be done very short.

The maximum curvature on a large portion of the line was 16° and on such sections a large percentage of the curvature exceeded 12°; the maximum elevation of the outside rail on curves was 6 in.; and as the speed of express trains frequently exceeded 40 miles per hour, the wear of the ball of such rails was very rapid, rendering their frequent change necessary. On those portions of the line where the grade exceeded 2%, the track was laid with rails weighing 65 lb. per yd.; on other portions rails weighing 56 and 60 lb. per yd., were used.

The equipment was fairly heavy, the largest engines carrying 45 000 lb. on each of four driving axles and 24 000 lb. on the truck. The freight cars had a capacity of from 60 000 to 80 000 lb., and the passenger equipment was the heaviest that was operated on the Santa Fé System at the time. To support the light rails under such loads, from 2 900 to 3 200 red spruce and yellow pine ties were used per mile of track, with tie-plates on all the curves.

Experienced track engineers will readily understand how serious a complication creeping rails was in the constant struggle to keep trains on the track under such conditions.

Sixteen years ago the speaker left the railroad referred to, at which time the trouble that resulted from creeping rails had not been entirely overcome. He is convinced that this was due to the poverty of the Company, which prevented the general application of the plan that was finally adopted to deal with it. This plan was not a complicated one, as it consisted of having sections of angle-bars rolled and cut into short lengths, about 5 in., or long enough to stagger the slots in the base of each pair of short angle-bars to correspond with the standard angle-bars and suit the punching in a joint tie-plate. These short angle-bars were bored for a track-bolt, and the rails were bored with a track drill to permit the application of two or three pairs of the bars to each rail; by this system each rail was firmly anchored to four or five ties, and the result was most gratifying. The speaker is of the opinion that if the number of ties to the mile of track is increased as the average size of the ties decreases, if every tie is tie-plated, if all spikes at the joints are driven in the slots in the angle-bars, and three short angle-bars are attached to each rail in the manner outlined, the creeping of rails on the ties will stop.

It might be added that if angle-bars with wide flanges, that permit the punching of a perfect hole in the flange, instead of a slot, are used for both the standard and short bars, a firmer anchorage of rails to ties will be secured than is possible with slotted angle-bars. If creeping track takes the place of creeping rails as a result of the more secure fastening of rails to ties, the situation will become serious and some of the other expedients mentioned by Dr. Waddell will have to be adopted.

The trouble the speaker has experienced resulted from the rails creeping on the ties, and as the track was laid with broken joints, the joints in the rails on one side of the track being opposite the centers of the rails on the other side, and as four-hole angle-bars and suspended joints were used, and the spikes at the

joints were driven in the slots in the angle-bars, the two ties near the joints were slewed away from their proper right-angled position in the track, and this movement loosened the ballast around such ties, narrowed the gauge of the track, distorted the alignment and surface of the rails, and invited trouble of various kinds.

LOUIS C. MARBURG,* ESQ.—The speaker is not a railroad man, but he may be able to furnish some information relative to a special shape of tie used to some extent abroad. The claim is made that it overcomes tie creeping successfully.

The invention in question does not attempt to prevent the creeping of the rails on the ties, but presupposes that some method can be found, which will prevent such creeping. If a rail is fastened solid on a tie, then two forces will act on the tie when a train passes over it. One force is due to the weight of the train and acts vertically downward. The other is due to wave motion (or whatever else may be the cause), and is in the direction of the motion of the train. With a horizontal track, the two forces are at right angles to each other, while on grades they are approximately at right angles to each other. The direction of the resultant force will form an angle with the vertical which will vary with train weight, grade, track, and operating conditions.

With the shape of tie commonly used, the forces mentioned must be resisted by two surfaces; by the bottom surface of the tie, which must resist the vertical component, and by the front surface, which must resist the component in the direction of the track. The bottom surface of the tie can be packed so that it can take care of the vertical component. The front surface, however, cannot effectively be packed.

The foreign practice referred to consists in the use of a tie having a surface at right angles to the resulting force acting on the tie, and incidentally a surface which can be packed tight. This is done by slanting the bottom surface of the tie as shown in Fig. 2. It may be said that the exact direction of the resulting force cannot be determined; it may also be said that this direction varies with different train weights and train speeds found on the same stretch of road, or even while one and the same train passes over a certain tie. Granting all this, however, it would appear that a tie of the general shape here mentioned, will show little tendency to creep.

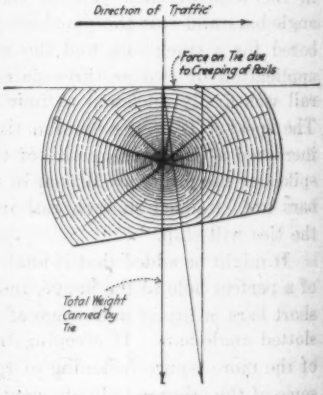

FIG. 2.

It should be added that while, of course, all ties are slanted the same way where the traffic is always in the same direction, on a single-track road where the direction of traffic alternates, it is the practice to slant some ties in one direction and others in the opposite direction, so as to prevent creeping either way.

* New York City.

A. W. Buel,* M. Am. Soc. C. E.—Engineers are indebted to Dr. Waddell for the preliminary investigation of a subject worthy of further research which might be carried on to advantage by co-operative work, to include the American Railroad Engineering Association. The data presented in the paper and otherwise available are hardly sufficient to support any definite conclusions and, possibly, the discussion, at least for the present, might profitably be confined to questions concerned with plan and scope of future research.

The speaker offers the suggestions that, first, statistics be compiled giving accidents and losses and proportion of spreading rails chargeable to creeping; and, second, that a number of sections of widely distributed track, representing the various classes of traffic, roadbed, grade, curvature, climate, etc., be equipped with the best, selected anti-creeping devices, and accurate records kept to determine cost of installation and, particularly, effect on cost of maintenance. Evidently, this programme would require a period of years.

That creeping is a cause of spreading rails has long been recognized. That it is the chief cause has not yet been demonstrated conclusively. Nosing of trucks and, especially, locomotive drivers, and rotten ties, have heretofore been considered a prolific cause of spreading rails. Data on the proportionate cases of spreading rails on tangent and curve, on heavy and light grade—with cases due to nosing or to rotten ties segregated where possible—should be of some assistance.

The speaker has heard no estimate of the cost of installing efficient anti-creeping devices. If we assume $600 per mile of track, which under present conditions will hardly be considered too high, it would appear evident that, for general application, the capital charge, running into nine figures, would be embarrassing. What is more important, the burden of proof is on the anti-creeping devices to justify the expenditure by demonstrating commensurately increased safety and reduced maintenance costs.

The speaker knows of no data bearing on these points except, perhaps, that some indications may be drawn from the effect of rise and fall on cost of maintenance of way, since some evidence has been presented that grade has an influence on creeping and, therefore, the probability that a part of the increased cost of maintenance on heavy grades is due to creeping.

Following Wellington's analysis, with consideration of constants proposed by W. L. Webb and J. B. Berry, Members, Am. Soc. C. E., and studies of special maintenance cost records on the Raton Mountain and Cajon Pass grades of the Atchison, Topeka and Santa Fe and the Soldiers Summit grade of the Denver and Rio Grande, an estimated cost of maintenance of way made in 1913 for the late Virgil G. Bogue, M. Am. Soc. C. E., by the speaker, for conditions on the Canada Pacific Railway west of Dunmore, showed that a 1% grade (26.4 ft. rise or fall per mile) increased the cost of maintenance of way by about 3%, or $81 per mile of track per annum, the base rate for straight and level track being 33.25 cents per train-mile and, for the traffic considered, amounting to $2 700 per mile per annum.

Under present conditions, the annual maintenance cost per mile, for an extensive mileage with average traffic, might not be more than $60 and might be

* New York City.

as low as $40 per mile for the increase due to a 1% grade, only a part of which can be assigned to increased creeping due to grade.

This, of course, has only an indirect bearing on the subject and is inconclusive, but indicates some reason for doubt that general use of anti-creepers can be justified on economic grounds. As to whether they can be justified on grounds of greater safety depends on what may be shown by statistics regarding accidents directly chargeable to creeping, and on cost of insurance by installation of anti-creepers.

Reference has been made to the resistance of the ballast to displacement of the anchor ties, in which connection it may be of interest to mention the triangular ties introduced by the late James J. Hill, F. Am. Soc. C. E., on the Great Northern Railway. The Principal Assistant Engineer of this road, Oscar S. Bowen, M. Am. Soc. C. E., writing for Alexander Steward, Assistant Chief Engineer, since deceased, "in regard to cross-ties of triangular cross-section" under date of May 13th, 1914, says:

"I have had good opportunity to observe the services given by these ties in Montana, Idaho, and Eastern Washington, where they have been and are being extensively used on this System. They are proving satisfactory to the Company and are being placed in large quantities every year. In order to get the best results, it is necessary that this style of tie be used continuous and not mixed with ties of other cross-section. Also, it is necessary to have good ballast and a well drained roadbed. I do not think they are as good as square ties in mud track or with poor ballast, and if used at intervals between square ties the tendency is for them to settle and transmit too great a load on the adjacent ties. When used continuous, however, they are giving the best of satisfaction and make excellent riding track."

The Percival reinforced concrete tie has a cross-section approximately triangular for the greater part of its length. It is very light, particularly in reinforcement. Twenty-five of these ties put in at McKees Rocks on the Pittsburgh and Lake Erie in 1906 remained for twenty months; twenty put in at Edgewater, Tex., in 1908, and ninety-six at Bayou Sale in 1910, both on the Galveston, Harrisburg and San Antonio Railroad, are reported "still in track, 1919". It was also reported that these ties could not be used between rectangular ties but, as in the case of the Great Northern Railway, must be used continuous.

The speaker attempted to compute the stresses in this tie with the result that, on the basis of vertical reactions, it seemed incredible that they did not fail under the first passing train, their moment of resistance being so deficient. When, however, reactions were taken normal to the surfaces in contact with the ballast, and conjugate pressure or passive resistance of the ballast considered, it became clearly apparent how they worked. This conjugate pressure theory also explains why triangular ties do not work well between rectangular ones and may have an important bearing on the subject of creeping and anti-creepers.

The speaker's experience inclines him to believe that, from a consideration either of safety or economy, or both, questions of supply, cost, life, and renewal of ties are of such great importance and offer such great returns for research and experiment, that they should not be crowded into the background by problems of lesser importance. At present prices, tie renewals are costing, in the more populous sections, from $600 to $1 000 per mile of track per year, the larger figure for inside track of three and four-track roads. However, when it can be

done without side-tracking more important work, a thorough investigation of creeping rails may be very profitably undertaken.

When the Special Committee to Report on Stresses in Railroad Track was first formed, and again in discussing* its last progress report, the speaker suggested that ties of triangular cross-section be included in the programme. He hopes it will find the means to do so before the work is finished, as this line of investigation may yield some interesting results and will probably be useful in the study of anti-creepers.

BILLINGS WILSON,† ASSOC. M. AM. SOC. C. E.—Preceding discussions have suggested the radical thought that there may be considerable economy in the adoption of a type of rail fastening and section of rail similar to the chair and "bull head" rail now in use on certain European railroads.

The present types of American rail sections and track fastenings were developed about forty years ago, at a time when wheel loads were about the same in this country as in Europe. For the light rails then required, the American T-section and spike fastening were more economical of metal than the European "dumb-bell" section and chair fastenings. In latter years, American car wheel concentrations have increased much more rapidly than those of Europe, but locomotive weights for high-speed machines, which are chiefly responsible for the heavier rail sections, have not varied so widely from European practice. We now find 120-lb. rail in use on American main line track, while 70-lb. rail is used in England where the chair type of fastening is required by law on all high-speed running tracks. In this connection, it has occurred to the speaker that there might be some economy in the gross weight of metal per unit length of track through the use of a lighter rail section and a heavy type of rail chair, that would give some lateral, as well as vertical, support to the head of the rail.

Economy in track maintenance through partial or total elimination of creeping is suggested by this type of fastening on two counts: First, as suggested in Answer 16-*I* of Dr. Waddell's paper, by supporting the rail under the head instead of under the base; and, second, by the action of the wedges which hold the rail in the chair, which if driven in the direction of traffic would tighten the rail against the chair as it starts to creep, and by thus increasing the friction between the rail and the chair serve to fasten the rails more rigidly to the ties and cause the creeping forces to be absorbed by the ballast. In other words, this wedge action would give the effect desired in Answer 16-*I*, just referred to, of anchoring the rail to each tie "so that the resistance of the ballast against being displaced by these anchored ties will equalize the tendency of the rail to move forward under rolling loads."

In the chair type of rail fastening, it is customary on double-track roads to drive in all wedges in the direction of traffic. On single-track roads wedges in alternate chairs on each rail should be driven in opposite directions. As a practical illustration of the action of these wedges under traffic, the speaker has seen on single-track lines as many as four wedges loosened and dropped out of place in as many rail lengths by the passage of several heavy trains in one direction, all of them wedges that had been driven in the opposite direction to this traffic. On

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1581.

† New York City.

double-track lines ejected wedges were rarely observed. Such as were found had apparently been driven in too lightly and were probably loosened, not by creeping, but by vibration in the track.

Summarizing: Reduction in cost of track maintenance by the elimination of creeping is suggested by the use of a rail chair with a wedge fastening which will tighten under traffic and anchor the rail to the tie; and reduction in cost of track construction by reduction of gross weight of metal per unit length of track, through the design of a lighter rail section and a heavier fastening, that will give some support to the head of the rail.

In opposition to the speaker's opinions, attention is invited to the fact that the rail chair is now in common use only in England. On the Continent, noticeably during the last three years, it is being gradually replaced by the T-rail with screw or cut spike fastening—standard American practice. The heaviest type of T-section in use is about 60 kg. per m., equivalent to about 120 lb. per yd., and replaces the 35-kg. rail of "dumb-bell" section. The "dumb-bell" rail is probably easier for the mills to roll, and, on account of its thicker web, presents less difficulties in cooling the section.

HERBERT C. KEITH,* M. AM. SOC. C. E.—In considering the creeping of rails there should be an appreciation of the fact that there are at least three different classes of causes of creeping: (*a*), temperature changes; (*b*), wave motion of the rails; and (*c*), friction exerted by the wheels on the tops of the rails.

(*a*).—On heavy grades, even with no traffic whatever (especially when the temperature changes are great), there will be more or less creeping, because expansion and contraction cause a movement which naturally exerts itself in the direction favored by other forces. Thus, expansion of rails, which on level ground would be entirely counteracted by subsequent contraction, becomes cumulative with successive changes of temperature to carry the rails down hill if the grade is heavy.

(*b*).—The extent of wave motion in rails and the resultant creeping are affected very materially by the amount and weight of traffic and by the vertical flexibility of the rail (which, in turn, is influenced by its strength and height and by the rigidity of the ballast and the firmness of attachment of rails to ties and of ties to ballast). Under ordinary conditions, the creeping of rails due to this cause (if not modified by combination with other causes) is in the direction of the traffic on a double-track line, or of the heaviest traffic on a single-track line, because the wave is continually thrust forward by the movement of the rails on the ties.

(*c*).—When a train is climbing a steep grade or accelerating its speed, the traction of the driving wheels of the locomotive tends to push the rails backward by a force equal to that given to the train for its forward movement. In like manner, when the train is coasting down hill with brakes fully or partly set, the friction of the wheels on the rails has a forward effect on the rails equal to its retarding effect on the train. As the backward thrust in climbing a hill and the forward thrust in coasting down hill (the chief examples of this cause of creeping) are both pushing the rails down grade, the movement of rails from this cause is generally down hill.

Often two or more of these classes of causes of creeping of rails are operating together, or in opposition, in a single stretch of track, with the resultant creeping

* New York City.

amounting to the algebraic sum of what all the forces would produce if acting separately. Perhaps some of the differences in the replies sent to Dr. Waddell can be accounted for by a lack of appreciation of the various causes of creeping.

The thanks of the engineering and railroad world, and, through them, of the traveling public, are due to Dr. Waddell for collecting these data, as well as the opinions of so many men so well qualified to discuss the subject, and for presenting the result to the Society, with his own conclusions drawn therefrom.

JOHN LUNDIE,* M. AM. SOC. C. E.—The question of creeping has been discussed from an academic standpoint, and also from the viewpoint of the mechanical engineer. Both classes of discussors assumed that the rail could be firmly held to the top of the tie, and the speaker would like to ask how this can be done. The creeping takes place between the bottom of the rail and the top of the tie, in other words, between the spike and the tie. If any method whatever of attaching the rails to the tie can be suggested, the speaker would be glad to know of it.

The question of creeping of railroad rails has been handled so well by Dr. Waddell in his valuable paper, that little can be added in seeking to determine the causes for rail creeping or the troubles engendered by such creeping.

When a physician is called in consultation on disease, if he is a truly scientific man he will first take cognizance of the condition and symptoms of his patient, tracing the history of the case and the probable causes for the trouble, thus obtaining a correct diagnosis of the case, before applying what he considers to be desirable therapeutics to combat the disease. He would probably see to it that the ordinary organs of the body were stimulated or inhibited so that they would function normally; he would seek if possible to remove the cause of the specific disease and ultimately attack it by knife or drug and prescribe a preventive against its recurrence. This, in the healing art, constitutes a parallel to the scientific method which should be followed by the engineer in seeking to cure the disease of rail creeping on railroads.

Dr. Waddell has ably diagnosed the case in so far as he has secured a mass of information as to the cause and effect of rail creeping. It seems unfortunate that not a single engineer interviewed has presented facts which would enable one to compute the amount of potential creeping which should be provided for on a new railway line. For instance, if engineers had definite information in some specific instance of the nature of the roadbed, the gradients and curvature, the weight of the rail, the type of joint, the temperature range for which rail expansion was allowed, the traffic tonnage passing over the line, the type of locomotives used, the necessity for braking on the stretch under consideration, and any other points which might be of value in the determination of rail creeping, then, in addition, if they had the actual amount of creeping which had taken place at different times of the year, under varying temperature and shade conditions, they would be in a position to make some computations which might be of value as applied to similar and even to modified instances of rail creeping or tendency to creep. It may not be the function of the Society to institute such investigations, but there is a Society—the American Railway Engineering Association—which might be induced to obtain such necessary data.

Apart from a consideration of the causes and effects of rail creeping, engineers are confronted with the fact that, under certain conditions, rails creep, and, in

* New York City.

creeping, may do a great deal of damage; thus, the immediate problem is either to stop this creeping or to provide for it so that its damaging effects may be reduced to a minimum. Heroic methods have been suggested by Dr. Waddell for the prevention of rail creeping and, undoubtedly, there are situations where drastic measures to prevent such movement would be justified. Probably no two rail creeping situations are exactly alike, although it is pointed out in the paper that comparatively simple anti-creepers or rail-anchors have given good service in checking such movement.

In the consideration of efficient means for the prevention of rail creeping, one should bear in mind just what the action of the rail is in its movement. At any given spot on a railroad, where and how is the creeping apparent? The rail certainly slips on its base between the head of the spike (or hook) and the tie, the tie-plate, or the chair on which it rests. There is every opportunity for rails to creep under ordinary conditions of spiking; indeed spikes are hardly ever in close contact with the base of the rail after a limited amount of traffic has passed over the track, even though such spikes may have been driven or screwed tightly in place before the traffic movement. Indeed, many able engineers advocate allowing a slight play at all times between the spike head and the base of the rail, claiming that tight spiking produces pumping ties and deterioration of track due to the wave motion of the rail.

Apart from the heroic measures proposed by Dr. Waddell, it is apparent that any form of rail-anchor must grip the rail and resist its longitudinal creeping movement by a counter action against a part of the track structure below the base of the rail. Admitting the rail must be gripped in some manner (preferably by neither boring nor slotting it), rail-anchors naturally divide themselves into three classes: First, some method of binding the base of the rail to a tie-plate or chair; second, some method of abutting the anchor against the tie; and third, some method of burying the lower portion of the anchor in the ballast or roadbed as indicated by Dr. Waddell in Paragraph 6 of his "General Conclusions."

Where rail-anchors are attached to tie-plates or chairs, the longitudinal thrust is communicated through the spikes to the ties and thence to the adjacent ballast and subsequent ties. The tearing open of spike holes is the principal objection to this type of anchors. Where anchors abut against the tie, such abutting portion usually consists of a slight finger or surface engaging the tie and, when the bearing of its finger or surface is near the top surface of the tie, there is a tendency to cause the tie to roll. It would seem from the consideration given by the writer to this problem that a satisfactory rail-anchor should grip the rail and be buried in the ballast, providing ample bearing surface to oppose the tendency to movement of the rail. Between such bearing surfaces and the adjacent ties, the ballast should be tamped tightly, and just as tightly in the line of the rail between subsequent ties. To bear against one loose tie alone would simply cause that tie to slew, and might create a situation where the cure was worse than the disease. In other words, so long as the rail has a tendency to creep, a rational simple method of counteracting this tendency is through a succession of ties between which the ballast is tightly tamped under the line of the rail.

It is almost needless to add that a sufficient number of anchors should be applied so as not to throw too great a strain on any one of them. They should

also be applied so that they do not interfere with the temperature expansion of the rail. If a single rail is supposed to expand only in one direction by virtue of slotted bolt holes then the natural place to place anchors on a rail is near the end which has unslotted bolt holes. If the rail is intended to have free expansion in both directions by virtue of all joint holes being slotted, the anchors naturally should be placed in the middle of the rail.

The merit of any rail-anchor should be judged by its efficiency in preventing rail movement and not by its price or ease of application, as is too often urged on the plea of economy of a very doubtful nature.

F. W. SKINNER,* M. AM. SOC. C. E.—Although the speaker is not qualified to submit an expert opinion in this practical discussion, he would like to make one or two academic remarks and to express his hearty appreciation of Dr. Waddell's pioneering in this subject which, although so old, has yielded so few records and so little literature. In that respect, the speaker is, by reason of large experience, particularly appreciative of the rational manner in which the author has approached his subject, which should serve as a model for the consideration of a wide range of important questions. The manner in which the data have been secured and the admirable conciseness and arrangement of the text, appeal to the speaker as valuable and desirable in a paper of this nature.

As regards the deductions, they are rational and logical, and where the author has appreciated the principal necessity of accurate observations and measurements of the creeping, for the purpose of more complete and decisive analysis, and the detailing of preventative measures, he cannot be wrong, no matter what facts may be established or what practice may be developed eventually.

Both this Society and the American Railway Engineering Association might well give further consideration to the subject, and authorize and encourage its study. In one particular, there is a very simple and obvious solution of this problem, namely, a rigid connection of the rails to the ties over a long continuous length of track. If an unyielding and immovable connection between each rail and each tie can be made, and the ties are then embedded in substantial ballast firmly tamped against them, it is only a question of how long a length of track must be for its resistance to overcome all creeping tendencies. If that is practiced, any system of anti-creepers or anchorages, which will satisfactorily connect the rails and the ties, will be successful.

Replying to Mr. Lundie, or rather suggesting offhand some recognition of his question as to how it is possible to secure the rails to the ties, the speaker would merely intimate that the great progress made within the last few years in the preparation of deformed bars for reinforced concrete suggests to him a possibility at least of rolling the rails so as to provide on the bottom flange some sort of projection, lock, or anchorage that would either grip a chair or a tie, or afford some positive bearing for one or both sides of the tie. Some such provision is well within the abilities of the rolling masters of to-day.

JAMES E. HOWARD,† ESQ. (by letter).—The creeping of rails is recognized as a matter of importance and it is especially interesting to have the opinions of

* New York City.

† Engr. Physicist, Bureau of Safety, U. S. Interstate Commerce Commission, Washington, D. C.

different observers brought together. From one point of view attention may be directed to the influence which creeping has on the fracture of rails.

It is noticeable in the records that a considerable number of rails that have failed, have been in service for long periods, escaping other vicissitudes, but eventually succumbing by the development of lines of rupture across the bolt holes. Such failures display no appreciable enlargement of the holes in respect to ductile flow of the metal, although some abrasion may have taken place, perhaps accompanied with upsetting at the edges of the holes.

Drifting tests of adjacent bolt holes have shown their ability to endure enlargement in diameter without material, if any, impairment of ductility. This difference in behavior between metal at the bolt holes across which rupture has passed and neighboring holes calls for explanation. The destruction of ductility in steel is in general accomplished in one way, and without recourse to heat, in one way only. It is the result of repeated stresses of some kind.

Considered as a beam, it will be noted that longitudinal shearing strains reach their maximum in the vicinity of the bolt holes. Due to position, the web of the rail in this part of its length is most strained by longitudinal shearing forces. The interruption of the metal of the web by the presence of the bolt holes must be an intensifying factor. These forces, although not in themselves, or probably not, competent to cause rupture, may aid in promoting it in conjunction with other conditions which affect this part of the rail.

The more pronounced influence is believed to be the hammering action between the web of the rail and the shank of a track bolt which embrittles the metal at the sides of the bolt holes. When clearance is exhausted at the sides of the bolt holes by creeping of the rails or otherwise, this hammering takes place. Anchoring the rails by spiking the splice bars to the ties necessarily brings the web of the rail into contact with the shanks of the bolts, causing this hammering action.

The wave motions of the rails are interrupted at the joints, accentuating the functions of the splice bars, which reach a maximum degree of efficiency only in providing resistance against vertical shear between the rail ends. In other respects, the efficiency of the splice bars is doubtless somewhat less in degree.

The forward impulses given the rails, which result in creeping, probably bear a relation to the longitudinal strains in the base. The driving wheels of the engine tend to force the rail bodily backward while the wheels of the train have the opposite tendency to crowd the rails bodily forward. Just what takes place in different parts of the length of a rail with the wheels in different positions is a matter of much interest, and one difficult to ascertain accurately. The major movement of the rail is downward and has been measured from time to time. The horizontal components and the manner of their distribution with reference to a fixed point on the roadway, as far as is known, have not received attention.

In respect to the magnitude of the forces with which an efficient anti-creeping device must contend, it might be said that it would depend on the rigidity of the device. The intensity of the forces which prevail in the longitudinal extension and compression of the rail are measured in thousands of pounds per square inch in the steel. However, these strains are exhausted within short distances. It is hardly conceivable that a close approach to the total arrest of longitudinal strains would be attempted or considered feasible to accomplish.

The place and manner of attachment of anti-creeping devices may be referred to. Probably a distributed effort is the most satisfactory one and experience with anchors attached to the rail flanges by frictional contact has not developed serious disadvantages, considering the safety of the rail. Indentations or incisions in the flanges ultimately lead to rupture and present elements of danger. In a comparative sense the cushioning effect of a rail anchor against the side of a tie is a mild affair. The modulus of elasticity of soft wood, particularly when taken crosswise of the grain, is only a fraction of that of steel, therefore anchors which are used against the sides of the ties individually add only a limited longitudinal restraint when acting within short distances. Each train disturbs the ballast in some degree, and the resistance of the tie itself thereby is reduced.

Ties would be expected to yield to individual impulses within the range of elastic longitudinal strains in the rail without interposing much resistance. Cumulative impulses would eventually move the tie, but the resistance of the tie against movement parallel to the track depends in a measure on the wheel load on the rail at the time. The firmness of the ballast below the rail is doubtless a factor in the case. It will be remembered, however, that longitudinal strains in both the base and the head of the rail are alternately tension and compression, hence impulses having to do with creeping, as far as the relations between longitudinal strains and advance of the rail are concerned, are the resultant effects of opposite strains.

Tests made by the writer in 1895 on the strains in rails in the track, including observations on the depression of the track on different kinds of ballast, showed the deepest depressions with cinder ballast, less depression on stone, and least on gravel. Frozen gravel ballast showed less depression than unfrozen gravel. These results were obtained on representative sections, in order to determine the relative depression of the rails laid on these three kinds of ballast.

It will be recognized that permanence of position of the rails in space depends largely on frictional resistance. Rails with or without anchors or anti-creeping devices remain nearly stationary by reason of the preponderance of frictional resistance between the rails and the ties, in excess of that between the rails and the wheels. The track partakes in a slight degree of the characteristics of a tread-mill. If the rails rested on anti-friction bearings the forward movement of an engine taken alone would be attended with a backward movement of the rails, accelerating forces, positive and negative, maintaining an equilibrium, the rate of speed of the engine in one direction and of the rails in the other depending on the laws of mathematics which govern accelerating forces. Of course, such hypothetical conditions could not prevail, but it will be recognized that the tendency of the motive power is to crowd the rails backward, while the wheels of the train tend to force them forward. The weight of the engines, the weight on the drivers, the distribution of these weights, and the effect of the wheels of the pilot truck, in a strict analytical sense, all appear to be of some importance.

A depression in the track which involves an almost equal depression of the adjacent roadbed necessarily draws the rails downward into the depressed area. The absence of tensile resistance in the ballast results in its separation, followed by recovery when the weight of the train is removed. The writer has found the area

of depression under some conditions to be of measurable degree at a distance of 30 ft. from the engine.

In respect to the bodily movement of the rail, observations appear to be needed to judge of the relative displacements and the resiliencies which occur when a forward impulse is given the rail by the train and the elastic reaction which comes from the ties. Elastic recovery in compression is a factor of the case. In vertical recovery, ballast and ties act to the best advantage in respect to resilience. In a horizontal direction it is not clear what efficiency the ballast shows in recovery against slight longitudinal displacements or a succession of displacements, each of limited extent. Only in the case of frozen gravel ballast would horizontal resistance and resilience seem to be of marked degree.

One of the most striking features brought out by the series of questions which were submitted to the railroads is the dearth of accurate knowledge of what creeping actually consists.

C. H. CHAMBERLIN,* M. AM. SOC. C. E. (by letter).—This subject is of great interest to all who are concerned in the maintenance and operation of railroads. The author is to be congratulated on the widespread interest which his paper has aroused, and deserves the thanks of the Profession for the form in which it is presented, compiling, as it does, competent evidence from so many sections of the United States.

It is not surprising to find some seemingly conflicting testimony. The conditions in one case are quite likely to differ sufficiently from those in another case as to seem to justify a different conclusion, and yet it occurs to the writer that, if all the facts in both cases were investigated and studied by the same mind, the conclusions might not be so much at variance after all.

The writer has never had the spare time to conduct a continuous and exhaustive scientific series of investigations and experiments to prove his theories in regard to the causes of creeping rails, but has had many years of experience in observing and combating this enemy to good track. He fully agrees with the author's conclusions that "the fundamental trouble is traffic", and that under traffic wave action in the rails and friction between the rails and wheels of moving cars and locomotives, particularly when accentuated by the application of brakes, are the causes which are to be treated and corrected. Anything that can be done toward stopping the one and lessening the effect of the other is proper treatment.

These points have been so tersely stated and yet so comprehensively covered in the paper that it is difficult to add new matter.

Wave Action.—The writer believes that a well-drained and compacted roadbed, with a reasonable sufficiency of suitable ballast, ties applied and maintained in accordance with the best practice, and with rails of a section and quality so suited to the traffic that there will be little, if any, appreciable deflection between supports, all properly spiked, jointed, and maintained, is as far as any one need go in most instances to reduce wave action to a negligible quantity.

It is appreciated that these precautions involve much, but they are all necessary to good track, pay in many ways, and are worth while where a saving can be effected. With many, the first cost is prohibitive, but any work, even done gradually, along the line indicated, will in time accomplish beneficial results. Engi-

* Fort Worth, Tex.

neers are rarely able to do all that they wish to, but are compelled to do something less within the funds allowed them.

There are localities where a compacted and non-yielding roadbed seems practically unobtainable, and it requires long years of constant effort in improving drainage, strengthening and compacting the roadbed by using materials best suited to the purpose, and adding to the ballast section, to show results. The semi-liquid earth forming the shaking prairies out of New Orleans, La., and the loosely deposited alluvium of Lower Louisiana are examples.

Brake Action.—There are many places where it is necessary to apply brakes to trains where the movement is always in the same direction. In many instances this is done so constantly and violently that no ordinary track conditions can withstand the longitudinal thrust imparted to the rails. The writer is acquainted with only one or two types of rail-anchors, or anti-creepers, which are helpful. These need to be properly applied, of course, and constantly inspected. The writer believes with the author that "deadmen", properly designed and placed, would be the most effective rail-anchors yet conceived.

Railroad men and some officials engaged in transportation service are not as a rule keenly enough alive to the evil effects of creeping rails, and it is believed that much good could be accomplished by well directed and persistent efforts to enlist their co-operation in handling trains so that brakes would not be applied more suddenly than is necessary.

In conclusion, the writer heartily subscribes to the author's hope that the American Railway Engineering Association will investigate this subject in all its phases. It is well worth investigating by every association interested in railroad maintenance and operation.

It is believed that when the subject is more thoroughly understood, it will be found that most cases will respond to some general treatment, but that each case must be studied individually to get the best results at the least cost.

VERNE LEROY HAVENS,* M. AM. SOC. C. E. (by letter).—A number of years ago, while Chief Engineer of the Tranvias de Mexico, the writer found that maintenance costs on the electric suburban line from Mexico to Tlalpan, were excessively high on that section of double track between Hospicio and Churubusco. Traffic was in one direction only, on each track. The track-walkers reported that the track spikes were very loose, and on investigation it was found that the ties were not perpendicular to the rails. The parallel highway was sprinkled frequently and part of the water ran on the tracks. Tall trees along one side caused so much shade that the ties did not become completely dry. Redriving the spikes continually in the damp wood soon enlarged the spike holes to such an extent that the wood would not hold the spikes.

The first step taken for correction was to open a small ditch between the highways and the track, allowing the water to reach below the coarse ballast. Authority was obtained from the Director of Public Works to cut many of the limbs from the trees so that the sun would shine on the track. The straightening of the ties was considered, but could not be done economically until the cause was removed.

It is believed that the creeping was partly due to the track (anchored at the

* New York City.

summits) being pushed down the hill by the horizontal component of the moving train. It is true that contraction tended to drag the track up the hill, but the maximum variation in temperature was probably not more than 50° Fahr. in the year, and the forces exerted thereby tending to lift the track and drag the ballast were insufficient to overcome the downward movement. It no doubt arrested the downward movement, but at the next period the creeping had a newly gained position from which to start, and continued at every opportunity, such as the vibrating passage of a train, the giving way of a spike, or the loosening of a bolt.

The rate of grade was 3 or 4 per 1 000, and the creeping was especially noted at the joints. On the upper section of the line the joints were as wide as the joint-bolts permitted, and it was feared that a great strain was being thrown on them with any great fall in temperature.

Walking down the track on the inspection trip, a place was reached where the joints were open less than on the upper section, and it was desired to remove the strain on the bolts by giving the track an opportunity to contract, if it would. A pair of switch-points were introduced at that place, on which the track could crawl up and down, and were left there for a month. The movement was so slight that the joints could be closed again by inserting a piece of rail. This place was taken as a point of departure for adjusting the lower section of the track, and was about 1 km. from the point, *P*, at the bottom of the sag, Fig. 3.

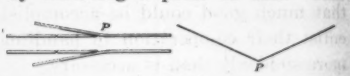

FIG. 3.

The expansion allowance was calculated on the basis of known data as regards the average mean temperature variation. These data are not now available, but the temperature change was actually somewhat higher than the measured difference, because it was proposed to trim the trees in order to leave the track more exposed. As nearly as can be determined from data now at hand, the desirable space for expansion and contraction totaled about 0.43 m. in the distance of about 1 km. The joints were nearly all closed, and it became necessary to remove the corresponding quantity of steel. Switch-points were placed at *P*, Fig. 3, and the rails ran out on them. After several weeks had passed it was found that in one track, one rail had crept out 55 cm. and the other rail 45 cm. In the other track, which was almost continually in the shade, the creeping was considerably less. This might be interpreted to mean that temperature change was the controlling factor, but, as a matter of fact, the traffic was one-way and it so happened that on the track partly in the sun, and which crept the most, the traffic was up hill.

After the extra steel was cut off and new holes were drilled, the rails were bent back and connected. Apparently, all the adjustment was not accomplished by the switch-points. When work began on surfacing and aligning the track, a considerable lift was made at one place and the track "kicked" sideways into line, the rail striking a laborer and injuring him severely. The lateral movement was very slight, but the force was great. This indicates that creeping can have a serious lateral, as well as longitudinal, effect, and pointed to the danger of removing large quantities of ballast from a short section of track on an extremely hot day.

The ties were then straightened and the spikes removed. The holes were partly filled with carbolineum, and oak plugs soaked in the same oil were driven into

them. New spikes were driven, bolts tightened, and the track was put in good condition. Although heavy suburban cars, as well as 40-ton gondolas, passed with a headway of 10 to 15 min., no further difficulty was experienced during the next two years. The only work done was by a track-walker, except perhaps picking up a low joint from time to time.

This experience with double track indicates to the writer that temperature variation is an important factor in rail creeping. It also indicates that the direction of traffic is not as important as the writer first thought. He had held the opinion that up-hill traffic tended to overcome some of the downward movement. To-day, he very frankly does not know what is the relative influence of the direction of traffic.

As to the problem of maintenance charges, the writer would use heavy bolts and err, if necessary, on the side of ample rather than scant expansion joints. This does not eliminate the tendency to creep, but by throwing more strain on the track-bolts, should mean better line and surface and less interference with traffic.

At another point on the same system, on an earth-ballasted steam line from the City of Mexico to Tizapan, it was proposed to place stone ballast on a short section of heavy grade in Mixcoac. The track ran from a bridge on a summit to a near-by sag; in the morning, suddenly, the track began to snap, that portion in the sag was lifted nearly 1 ft. off the ground, and trains could not pass. Several hours were required for it to settle back to place, and during this time men began at the top of the hill and pounded the joints as they traveled down, to free its movement and cause settlement. Whether or not this was effective, or whether the midday and early afternoon sun was the cause is not known, but it points out the fact that ballast must be plentiful in sags, even if it is not so plentiful on the summit. It is feared that on the double-track line too much expansion space for good practice, in the strictest sense, was allowed. However, it saved money. The lighter single track should have less space between the rails ends, to avoid lifting, but without doubt maintenance charges ultimately would be increased by creeping.

William E. Robinson,* Assoc. M. Am. Soc. C. E. (by letter).—The writer considers the Engineering Profession deeply indebted to Dr. Waddell for his valuable paper, which so clearly brings to light most of the causes of creeping of railroad rails; he believes, however, that the author rather emphasizes the hazard due to this when he states in his conclusion that, "Rail creeping * * * causes great destruction of railroad property and transported goods, and seriously endangers on a large scale the lives of the traveling public." In ten years' experience on maintenance-of-way work the writer knows of only one derailment attributed to creeping of rails, although any number of accidents, caused by improper gauge, splitting of switches, etc., may have been due primarily to this.

C. D. Purdon, M. Am. Soc. C. E., has touched (page 388), on an interesting phase of the subject which, so far, has received very little mention, namely, anchoring track on bridges or trestles. The writer agrees with Mr. Purdon in that the fish-plate is superior to any kind of angle-bar for use on bridges subjected to creeping rail. A short time ago the writer noticed a trestle bridge approximately ½ mile long, on which the rail had been creeping extensively, breaking the flange on about

* St. Augustine, Fla.

twenty-four angle-bars. Evidence on the ground indicates that these angle-bars were broken as follows: The creeping of the rail caused the angle-bars to push the spikes on the adjacent ties over and up, the corners of one end of the bars were then supported by the spikes which they had partly pulled and bent, and were presumably broken by the force transmitted by passing trains.

The writer knows of a case similar to that mentioned by Mr. Purdon, in which one rail crept in one direction and the other rail in the opposite direction. This happened on a bridge about 600 ft. long, consisting of two 1 200-ft. trestle approaches and a draw-span; the track on this bridge is tangent, the east approach descends on a 0.4% grade from the draw-bridge, and the remainder of the bridge is level. The south rail of this bridge creeps west and the north rail east. The writer expects to investigate and ascertain, if possible, the causes of this phenomenon.

B. H. Bryant, M. Am. Soc. C. E., mentions (page 400), the application of 5-in. sections of angle-bars bolted to the rail and spiked to the ties. The writer recalls seeing some of these sections a few years ago, which were applied on the Pennsylvania Lines East. It would be interesting to hear how these short sections of angle-bars compare with standard makes of rail-anchors as to cost and efficiency.

The railroad on which the writer is now employed follows the Atlantic Coast for several hundred miles. Very little trouble has developed from creeping rails, due presumably to the fact that it is a single-track line, with well drained roadbed, light curves and grades, no extreme ranges of temperature, and a traffic that is nearly balanced. What little rail creeping has developed has been on the heavier grades, the grades undoubtedly being the cause.

It is the writer's opinion that if the avoidable causes of creeping rail are removed, namely, poor drainage, narrow roadbed, insufficient ballast, and poor track construction, any track can be prevented from creeping by the proper application and maintenance of standard makes of rail-anchors.

Frank Reeves,* M. Am. Soc. C. E. (by letter).—The writer thinks that the thanks of the Profession are due to Dr. Waddell for the laborious collection of data on the subject of rail creeping which he has made. The paper will rank as a classic for all time.

Sufficient importance, however, has not been given by the author to the axle loads carried; none of the questions directly relates to it, and it is mentioned last of all the factors in Paragraph 4 of the General Conclusions. The results of the writer's experiments, detailed in a paper† read in January, 1918, before the Institution of Civil Engineers, London, England, showed that this was the predominating factor in the amount of creep produced. A lightly loaded wheel will produce no creep at all, because the resistants (anchorage) produced by the friction of splices, fastenings, ballast, etc., is sufficient to hold the rail; hence, creep was not troublesome in early railway days. A heavily loaded wheel, however, produces a creeping effort beyond the resistance of the anchorage, and "creep" is the result.

The writer has had an experimental ¼ mile of special track under observation since October, 1916. It consists of 100-lb. rails on hard wood ties with tie-plates. The rails are held down tightly to the ties by ⅞-in. through bolts and clips gripping

* Buenos Aires, Argentine Republic.

† *Minutes of Proceedings*, Inst. C. E., Vol. CCV (1917–18).

the foot of the rail, so that each tie is an anti-creeping device in itself. These bolts were tightened periodically until they refused to move any more. The stretch was chosen so as to be under the most adverse conditions, namely:

(1).—It is on a down grade (0.5 per cent.).
(2).—The traffic is all one-way, being double line.
(3).—The speed of the through trains is high.
(4).—It is subject to suburban as well as main-line traffic.
(5).—It is on the approach to a local station so that local trains apply brakes on it.

The maximum creep on this stretch has been about 1 in. in the 4 years, while the average is perhaps about one-half that amount.

The writer is of opinion that creep cannot be wholly eradicated under adverse conditions such as these, where axle loads are heavy, but that by a type of fastening on every tie, which can be tightened when slack, it can be reduced to an amount so small as to be unimportant.

The axle loads carried on the stretch mentioned were only about 40 000 lb.; with higher loadings, it is thought that the creep would have been appreciably more. The line is in low fill of good earth.

The writer's experience with "deadmen" as a preventative is adverse. The result is breakage of the anchoring rods, bolts, etc., and, what is more serious, the buckling or kinking of the track behind the anchorage in hot weather. In any attempt of this nature, the "deadmen" would have to be very numerous and extend over a long length of track to have any chance of success.

W. M. Robinson,* Esq. (by letter).—The most serious troubles engendered by creeping rails are as follows:

Distortion of gauge, produced by joint ties pushed out of normal position bunching those with which they come in contact, causing unequal tie support to the rail at such points; opening and closing the gauge alternately on curves, and producing on tangents irregular gauge and deformed alignment; elongation of expansion spacing to such extent that bolts are sometimes sheared and the continuity of the rail connections broken. This process of elongation changes to compression at some point in the vicinity of intersecting descending and ascending grades, and tightly closes all spacing between rail ends, thereby producing liability to sudden lateral movement of the track under moving trains, which in the past has caused many disastrous wrecks.

The writer inclines to the view that when maintenance is normal, creeping is, in a general way, approximately proportional to the amount of traffic passing over it. With the traffic in one direction only, the creeping is probably 60% greater on descending than on ascending grades.

On single-track lines the writer inclines to the opinion that, generally, creeping is greatest in the direction of heaviest traffic; however, careful consideration of the gradient in the opposite direction may in some cases modify this conclusion. A combination of conditions affect the creeping of rails on both tangents and curves and it is doubtful whether, generally considered, there is appreciable difference in amount.

* Valuation Engineer, Georgia Railroad Company, Augusta, Ga.

It is doubtful also whether, to any appreciable extent, the degree of curvature affects the creeping of rails; several factors enter into and more nearly determine the phenomenon, if it exists; notably, proper or improper adjustment of the track, grade percentage, and speed of trains.

It is a fact well known to practical trackmen, that in certain localities creeping is greater on one side of the track than on the other, and although various causes have been assigned, the most feasible probability appears to be inefficiency in joint fastenings and improper spiking on the side most affected. For the same train loads, gradient, and speed, creeping is greater on light than on heavier rails, the obvious reason being greater stability, and less wave motion of the rails on the part of the latter.

The rate of creeping is greater in sand or cinder ballast than in stone ballast, for the reason that broken stone affords firmer foundation support, and the friction between cross-tie and stone ballast offers greater resistance to the tendency of displacement of the tie by rail creeping.

The creeping varies with the nature of the road-bed, and with the kind and depth of ballast; on account of weight, the most effective barrier to creeping is a good foundation of broken stone, with the track closely filled to the top of the tie, well shouldered on the ends. The amount of creeping is affected more or less by the temperature and weather conditions, and the trouble is probably accelerated by the effects produced by expansion and contraction, during the hot days and comparatively cool nights of summer. The creeping varies to a marked extent with the character and efficiency of the rail splicing; ill-fitting angle-bars with faulty punching quickly provide means for undue opening in joints, and the wheels pounding at the ends of the rails greatly increase the tendency to longitudinal movement. The amount of creeping does not appear to depend on the use or non-use of rail chairs or tie-plates.

The most effective methods found for lessening or checking the creeping of rails, appears to be proper construction and maintenance, which requires the following details: Joints properly slotted and spiked, with the best type of rail-anchor placed opposite, and as many intermediates as appear to be necessary, broken stone ballast, smooth surface, good alignment, track filled to top of tie end to end, with shoulder sloping to road-bed.

The most potent causes incident to rail creeping appear to be wave motion of the rail in the direction of heaviest traffic, high speed, the effect of gravity, and friction between the wheels produced by sudden full power application of the brakes. No scientific computations were made concerning the resultant force causing creeping under specific conditions, and the writer doubts whether a dependable formula for doing so has yet been evolved.

F. T. Darrow,* M. Am. Soc. C. E. (by letter).—This paper is a splendid presentation of a subject which is of great interest and value. The writer was reminded in reading page 366 that on certain tracks which he has had occasion to watch, and even measure accurately from day to day, he had found the following:

First.—At Plattsmouth Bridge, the grade is 1% up the hill from each end of the bridge. The alignment is tangent over the bridge and to the east, and a 12° curve

* Lincoln, Nebr.

about 100° long on the west end. On both the old bridge and the present bridge the rails creep in opposite directions; in 1903 when new rails were put in the new bridge, the writer marked them with care at the fixed ends of the span and observed the creeping of the rails for several months. The north rail creeps east across the bridge, down hill to the east on tangent, and opens up joints on the inside rail of the 12° curve on the west approach. The reverse is true of the south rail. It travels west and tightens the joints on the curve. The rail is relaid on this curve about once a year and the same condition continues with each new set of rail, namely, the outside of the curve has tight joints and the inside has open joints.

Second.—Conditions at Nebraska City and Rulo are similar, with a tangent over the bridge to the east and a curve to the north on the west, though with easier curves and different grades. The general condition of open and closed joints on the curve, however, obtains.

MARCEL K. SESSLER,* ESQ. (by letter).—The author is to be commended for the variety and nature of the questions asked in this paper, as well as his choice of replies. It is noticeable, however, that he has refrained from criticizing any of the answers except in a very general way. It seems to the writer that were he to give a concise understanding of his views in conjunction with the replies which he received, it would be of real value to the Engineering Profession.

That rail creeping must be reduced, if not entirely avoided, is unquestionable. That it can be done only on a basis wherein economy is a most essential factor, is absolutely certain. The suggestions which the author has submitted for stopping rail creeping the writer believes to be economically impossible under actual operating conditions. They are mechanically possible of fruition provided sufficient funds are available to accomplish the desired results. In order to better the actual operating conditions, they must be made on an economical basis.

If the author's suggestions should be applied to roads, for instance, like the Denver and Rio Grande Railroad in the West, or portions of the Pittsburgh and Shawmut Railroad in the East, both of which contain very heavy gradients and curvature, the cost would be prohibitive.

The writer takes the liberty of emphasizing a thought which seems to have been given little or no attention in the replies received. From a technical standpoint, it is fair to presume that a body which acts as a unit presents a greater resistance than a similar body acting in individual parts. He refers now specifically to what many railroad engineers term "unit construction", where rail, tie, and spike remain together. Where rails are not effectively spiked to the ties, individual tie pumping results; the same action is not obtained if rails are so effectively spiked to the ties as to cause them to remain in close contact with the rail base. It appears, therefore, that the less efficient the spike, the shorter the wave length which induces creeping. In order to obtain a longer wave length, it would naturally require more rigid fastening of the rail to the tie.

R. EMMET KEOUGH,† ESQ. (by letter).—In considering this subject the writer accepts the hypothesis that the present form of track construction is to be continued

* Gen. Mgr., Am. Spike Co., New York City.

† Asst. Engr., Maintenance of Way, Canadian Pacific Railway Company, Eastern Lines, Montreal, Que., Canada.

for many years to come and, therefore, will attempt only to apply a remedy to the existing conditions. On the light, 60-lb., suspended joint track where he first started track work, he soon observed the trouble due to the displacement of the light ballast around the joint ties. Although in many cases this resulted in no ill effects other than causing low joints, due to the ballast disturbance, at other points on heavy grades it was found that the ties became bunched due to their continual movement in one direction, also that the rail gave evidence of kinking at the joints, resulting in an all-round poor condition.

After the longer, three-tie joints were introduced, whereby the two shoulder ties only were anchored, while the middle, or joint tie proper, was unanchored, trackmen soon noticed improvements around the joints and there are to-day many older trackmen who insist that the 44-in. joint is the best they have ever used. On going into this joint more closely, however, it has been found that although the longer joints have advantages over the shorter ones, they also have disadvantages, the greatest, perhaps, being the trouble found in their use on curved track. It has been found that the greatest advantage in this longer joint fastening is due to the fact that the middle, or joint tie, having no joint slots above it, is unanchored, thereby allowing freedom of rail movement over this tie without disturbing its seat in the ballast bed. It was this long joint which first proved to the writer that joints will do best when entirely unanchored to the ties.

There are at present many trackmen who favor the "hit-and-miss" method of laying rail, not only because it is cheaper, but because it is found that better joint conditions are had when the job is properly done; by this is meant that the rail in all cases must be anchored elsewhere throughout its length as quickly as possible after it has been placed.

Flexibility is a necessity in track construction in order to secure easy riding conditions. Reference need only be made to earth-ballasted track with its "center-bound" section to understand this; also cinders, either as ballast or sub-ballast, prove it. The much despised swamp or muskeg track certainly has the advantage over the more solid track in this respect; furthermore, it does not often heave with frost. Rail also will wear much longer over muskegs in a cold climate due to softer roadbed.

Owing to increased wheel loads and other causes, present-day rail is not lasting nearly so long in first-class service as that of earlier days. The writer finds it quite possible to anchor rail sufficiently well to enable it to be carried through its full life without any ill effects due to rail creeping. In order to do this, it is necessary (on old lines) before relaying to study each local condition in order to provide suitable means for taking care of the new rail. Most consideration must be given to the average expansion allowance per mile of track, instead of too much to individual joints. It is not necessary that each and every rail should contract and expand from mid-point equally in each direction in order to procure good track, but proper means should be provided for allowing the rail to move somewhat as a unit toward the joint where experience has proven that it is given to "bunching up".

As an example, consider a section of track carrying one-way traffic in which it is found that the rail is given to bunching at the foot of a grade where brakes are applied frequently. At all such points—and they are legion—it is practically impossible to lay rail too tightly at the summit, provided that allowance is made for

the "filling up" of expansion at the foot of the grade. All approaches to stations, water tanks, grade crossings, etc., must be treated in this manner if good conditions are to be obtained.

Main-line frogs and switches should not show signs of kinks, provided such allowances are made and a reasonable number of rail-anchors are applied as quickly as possible after the rail is placed. There are in many places swamps or muskegs where it will be found that even though every tie is anchored to the rail, creeping will not be entirely stopped. Only extensive reconstruction will cure such places, and it is usually found that such methods are considered too costly to be undertaken.

There are numerous cases on level, single-track lines where it is found that although the rail moves back and forth under trains, the traffic is so equally divided that no considerable movement in one direction occurs. At such points it is often noticed that the greatest difficulty is found in retaining portable rail anchors. At such locations it is generally best to anchor practically every tie if the movement is not entirely balanced, but where experience proves that it is equal no harm will follow if the rail is left entirely free to run to and fro a slight amount, leaving the ties undisturbed, provided all ties are plated to prevent abrasive action.

At all points where anchoring is necessary it should be done in such a manner that some freedom of movement in the opposite direction to the expected movement of rail is provided for, as it will be found that all track fastenings and ties suffer less if this is considered. Often on single track a heavy train will drive a stretch of rail tightly against the anchors, while trains in the opposite direction will only send it back a small amount. In other words the rail will be found to lay away from the anchor bearings entirely in many cases. The anchors should be on duty merely to arrest any unusual movement due to the sudden application of brakes, etc.

The ill effects of the positive type of anchor in joint slots of the old type, or even worse in the enclosed slot or hole in the base angle of joints, are known to all. With such anchorage, spikes invariably were found to lift gradually out of the joint ties. One who gets his nose close to the ground in such cases finds that this is due to the fact that, first, the spikes were overloaded with longitudinal rail movement and, second, that not enough room was allowed for the spikes in the joint slots. One side of the slot came in contact with one side of the spike at the under edge of the joint, while the opposite side of the slot made contact with the other side of the spike on the top side of the base of the joint. After these spikes have been applied for a sufficient time to allow some displacement, which will occur when the track is not sufficiently anchored, the spikes in the slots will be subjected to a continual pincers action. They will be pushed downward by one edge of the slot and pulled up again by the upper opposite side. This soon loosens the spikes and allows them to creep upward out of the tie and thus become ineffective as an anchor or rail fastener. Although it has never been considered good workmanship to drive spikes close into the corner made by the joint or angle-bar projection, nevertheless spikes so driven do not work up like these joint slot spikes.

Since it is generally conceded that the greatest difficulty with rail creeping is that due to heavy traffic, the writer believes that when furnished the necessary

rail, ballast, and appliances to correspond to the traffic handled, it should be possible, at least in the majority of cases, to wear out a rail before experiencing much difficulty through creeping.

C. F. YARDLEY,* ESQ. (by letter).—The author is to be commended for his thorough effort in making such a wide search for information concerning the creeping of railroad rails. The views obtained were from many sources widely distributed throughout the country and from those qualified to discuss the subject; every phase of rail creeping seems to have been covered, hence the variance of opinion expressed.

The writer has been intimately associated with maintenance of track on three large roads embracing the northerly section of the United States from coast to coast. It has been his privilege to have observed the creeping of rails under practically all conditions and he therefore can appreciate the majority of the views cited.

That rail creeping is a costly and serious condition is generally acknowledged, and the résumé of answers to the various subjects brought out in the questionnaire apparently covers the situation very well. The writer, however, is at variance with one conclusion advanced, that is, the method of anchoring rails under extreme conditions by the institution of "deadmen". It is his belief that each rail under all conditions should be anchored independently, thus maintaining a uniform resistance throughout the entire length of any particular section of track.

Contributing to the discussion, which has brought out some very interesting and vital features, the writer will consider first the effect on creeping of using joints without slots for spiking and maintaining uniform spacing of ties. It is his opinion that the rail should be given free movement and that the joints should not be used to check the creeping of rail. In practically all instances the joint ties under such conditions are loosened in the ballast and damaged through displacement of joint spikes, thereby creating a weak condition under that part of the rail which is considered the weakest portion of the track structure—the joint.

Furthermore, on account of the tendency of creeping rail to pull or push the joint ties, the uniform spacing of the ties is immediately interrupted. It is understood that the ordinary 24 or 30-in. angle-bar is considered in this connection. In the case of the base-supported joint this may not hold true, that is, insofar as the efficiency of the type of bar is concerned. If the angle-bar stands up and outwears the rail, it has done its duty, and this is generally the case when track is given the attention it should have.

In letting the joint "run wild", so to speak, the flange of the angle-bar should be flush with the base flange of the rail and not extend beyond the edge of the flange, and thus permit spiking through the ordinary tie-plate, using a larger spike if necessary. Of course, this may slightly weaken the angle-bar, but this weakening can be compensated for by adding a rib in the top of the bar or otherwise building it up. In such an arrangement, suitable and sufficient frictional rail-anchors should be installed on the base of each rail against the same tie in opposite rails to correct the creeping. If one side has a tendency to run more than

* Maintenance Engr., New York Div., New York, New Haven and Hartford Railroad Company, New York City.

the other, sufficient additional anchors should be installed on this rail to stop the creeping and prevent the slewing of ties.

This, in the writer's opinion, is the most economical and satisfactory method of anchoring. In this construction an extremely objectionable procedure and costly expense is avoided by eliminating the necessity of re-spacing ties under joints, which in many instances means the disturbing of practically all the ties in a panel, and destroying the general surface of the track. Of course, where large tie renewals are necessary (which condition, however, is being generally reduced or curtailed by the use of treated timber), the spacing can be done at the time of renewal at less expense than otherwise.

One feature of anchoring track should never be overlooked, that is, the advisability of installing anchors the same day that rail is laid and in sufficient numbers to hold the rail. There is apparently a disposition on the part of many engineers not to do this, probably due to causes beyond their control.

In the discussion it is mentioned that the manufactured anchors now on the market are not suited for single-track anchoring. It is obvious that the same type of anchor, reversed, can be used against the opposite side of the tie, although preferably on another tie to prevent the churning of ballast, and both may be necessary. Mention is also made of rolling the rail in such a manner as to provide an anchor at intervals throughout the rail base, the bearing face to be integral with the rail. If this were possible it would, in the writer's opinion, be a questionable and highly objectionable institution for the following reasons:

1st.—In laying new rail, owing to the fact that the anchors were in a fixed position on the rail, the necessity of spacing many ties in each panel to meet the anchor would be apparent. This procedure would disturb the general surface of the track, the track would be out of service for a much longer period, and the cost of laying the rail would be increased out of proportion, on account of the joints of the new rail running ahead or back of the location of the joints of the old rail.

2d.—If one rail travels faster than the opposite rail in creeping, the bearing faces on one side become useless or would put the ties out of square.

3d.—The bearing faces would not match on curves where the rails on the low side run ahead of those on the high side; many other complications would arise, principally around interlocking points where it is not so convenient or even possible to respace ties.

It is obvious, therefore, that a desirable rail-anchor would be one that could be detached and re-applied to any position on a rail. Unquestionably, the minimum creeping of rails will be found in track provided with good drainage, substantial ballast of sufficient depth, and a high girder rail commensurate with the tonnage handled and this should be the aim of every engineer in charge of maintenance, to the extent of even a gradual betterment process dependent on funds available to obtain this result.

The best track construction will require rail-anchors to hold the rail in check and they should be installed in such numbers and at such locations as will meet the requirements. It is a mistaken idea to expect any type of rail-anchor to overcome impossible conditions. This applies equally to rails, ties, joints, splices, and track fastenings in general.

The use of "deadmen" is a departure which in the writer's opinion should be avoided. This method of anchoring tends to arrest creeping by a positive stop, allowing the intervening rails to run at will, creating open and tight joints which might produce buckling under high temperatures besides disturbing the roadbed between anchorages. If sufficient "deadmen" were installed to anchor each panel or stop the creeping, the cost would be prohibitive and the funds could be used to better advantage.

It has been observed that in many cases a sufficient number of frictional anchors have not been installed to meet the various creeping conditions. If this matter were followed more carefully and intelligently by those in charge of track maintenance much of the trouble would be largely eliminated, if not entirely overcome.

As to the method of anchoring rails on large structures and draw-bridges, there are points where this has been done very successfully by placing anchors in sufficient quantity on the approach track at each end of the bridge, and using them also in the bridge floor. No adverse conditions have resulted, and the rail has been held.

The conditions arising from rail creeping came forcibly to the writer's attention while acting as section foreman on a single-track road in 1907, in the Rocky Mountain Section. The line in question had extreme curves and grades, the heavy tonnage had traveled down grade, the light movement up grade. The track structure consisted of rail of light section with angle-bars, native ties embedded in scattered cinder and dirt ballast, and no tie-plates.

About every three or four days the bolts in one or two joints would shear, making it necessary to insert between the rails so-called "Dutchmen"—small sections of rail 2 in. or more in length. These "Dutchmen" became so numerous that occasionally all would be taken out, the rail driven up and a short piece of rail 2 ft. or more in length placed to fill the gap, these operations involving considerable time of the gang. During one winter about 15 ft. of rail got away, and the foreman below who had the terminal end of the line probably suffered considerable inconvenience.

Experience and subsequent observation brought the realization that immense labor and material saving might result from the design of a satisfactory anti-creeper, and with this in view the writer has spent practically all his spare time during the past ten years in trying to devise a one-piece efficient anchor which would stand up under severe strain. Four years ago he installed a device only 2 lb. in weight. Although still under test, anchors of this design are giving good service and are standing up well; at the present time approximately 150 000 are in service.

BERNARD BLUM,* M. AM. SOC. C. E. (by letter).—The questionnaire sent out by Dr. Waddell in regard to the creeping of railway rails, and the replies received, as well as the author's conclusions, have very evidently aroused considerable interest, as shown by the numerous discussions which have been submitted.

In view of several discussions which the writer has had with railway engineers and manufacturers of anti-creeping devices who have had considerable experience with the creeping of rails, and some measure of success in coping with the trouble,

* St. Paul, Minn.

he believes that there are a considerable number who do not subscribe to all of Dr. Waddell's conclusions.

The author started his investigation by sending out a list of questions to the Federal Managers or the Presidents of seventy railroads on this Continent. It is apparent from the answers published that the men either did not understand the questions fully or that they did not know their subject. The contradictory replies received to so many of the questions justifies this rather severe censure. Some of the replies indicate that different interpretations were put on the questions and others that peculiar conditions existing at certain locations have prompted replies of a most general nature. It seems to the writer that the author should not have published answers that cannot be applied as a general truth. Certainly, the young engineer should not have thrust on him such statements as that contained in Answer *8-K*, where the following is given:

"This [creeping] is undoubtedly due to the fact that the heavier the rail the greater the expansion. One roadmaster reports that a 50-lb. rail will expand ½ in. where a 90-lb. rail will expand ⅛ in. under the same temperature variation."

Such a statement is untrue, and the two sentences are contradictory. The writer does not imagine that Dr. Waddell subscribes to such a belief, and therefore why publish it as of sufficient weight to have bearing on this subject?

The more elementary phases of creeping rails are of general knowledge at this period and, therefore, it does not appear necessary to call attention to discrepancies contained in the several answers. It is to be regretted that the author did not ascribe a letter to each of the roads which replied to his questionnaire, so that the same letter in each of the sixteen answers would relate to the same railway.

It seems to the writer that the "General Conclusions" are subject to some criticism. A rather broad statement is made that the creeping of rails seriously endangers, on a large scale, the lives of the traveling public. If creeping conditions are allowed to accumulate to such an extent it would seem that those responsible for the maintenance of the track are at fault, as there are means for anchoring the rail to prevent sudden catastrophes. Probably what is meant is that, if left unchecked, the creeping of rails would endanger the lives of the traveling public. If rail creeping increases with the amount of traffic, it would seem to follow that the amount of creeping would be in direct ratio to the traffic, assuming, of course, traffic in the same direction. However, the main conclusion would seem to be Paragraph 6, summarizing the author's method of preventing creeping.

The writer cannot subscribe to the use of "deadmen" or piles driven into the roadbed. Connecting "deadmen" with strong adjustable rods would create a rigid foundation at each point of contact, tending to fracture the rail. Placing two or three "deadmen" to each stretch of moving track would introduce considerable stress in the rails so anchored and introduce additional stresses in the joint fastenings. The intermediate rails would not be anchored and the intermediate joints would be stretched or compressed to the limit, which condition is one of the bad effects of creeping rails. The matter of cost has not been considered and it is probable that this would be very great. Experience and reason indicate that each panel of track should be anchored, and certainly the cost of "deadmen" as described would cause their rejection in favor of making the track truly first class.

The various opinions as to the effect of tie-plates on creeping are somewhat interesting. There are so many opposite views on this subject that it would seem that the various answers have been prompted by more or less accurate observations of several cases, and the conclusions to be drawn are rather indefinite.

A number of the answers seem to advocate the thorough anchorage of the rails at the joint fastening. Several of the discussions of the subject advocate the non-spiking of rail joints to the ties. For several years the writer has advocated the laying of steel without spacing the joint ties, and non-slot spiking the angle-bars. It is probable that we will never get away from some movement in the rail, and where the joints are slot-spiked to the ties the joint ties necessarily move backward and forward with the rail. Where the movement is greater in one direction, the joint ties are moved off their bed on untamped ballast. If the creeping has been checked, there is a slight movement of the ties which serves to cut the tie down vertically. As the joint is the weaker section of the track, the tie conditions there should not be made worse than at intermediate points. Furthermore, the stresses in the joint fastenings should not be augmented by introducing more work.

On the road with which the writer is connected, creeping of rails has been reduced to a point that it does not present a serious danger, and this has been accomplished by the liberal use of certain well-known rail-anchors. Several types of rail-anchors have been found which can be fastened to the rail so effectively that the rail does not slip through, and this has been indicated by the actual failure of a number of these anchors due to the fact that the engineers originally failed to place enough of them to take care of the actual stress in the rails. In Dr. Waddell's General Conclusion No. 6, there is expressed the necessity of evolving such a type of anti-creeper which, in conjunction with his suggestion of creosoted piles, would absolutely prevent all creeping.

Although various causes for the creeping of rail have been outlined in the answers to the questions and the subsequent discussions, there is little question in the minds of railway engineers as to the principal cause, namely, the wave action in the rail caused by the moving wheels, and the extent of this depends on the stiffness of the track structure and the solidity of the embankment and natural ground. As the drainage, ballast, ties, and rail conditions are improved, the creeping tendencies are lessened. All efforts should tend toward improving these conditions, thus rail creeping can be reduced to a point where well designed anchors between rail and ties will take care of the problem.

This subject is a live one, and it is probable that the managements of some of the railroads have not realized the necessity of adequate anchorage, and to the extent that Dr. Waddell's investigation has aroused them it will prove of considerable value.

J. A. L. WADDELL,* M. AM. SOC. C. E. (by letter).—Considering the almost vital importance of this subject both to the railroads and the general public, the volume of the discussions submitted is rather disappointing; but this can by no means be claimed concerning their quality, for they are unusually fine and comprehensive. The writer took the trouble to send several hundred letters and postal cards, requesting discussions, to members of the Society, the American Railway

* New York City.

Engineering Association, and the railway profession in general, hoping possibly to obtain thereby as many as fifty replies, but he has had to content himself with about one-half that number. It is true, as has been stated to the writer a number of times, that the answers to the somewhat voluminous questionnaire and the résumé thereof cover the ground quite thoroughly, but the discussions submitted prove that each one who wrote had important ideas of his own which were not treated thoroughly, or sometimes not at all, in the replies to the questionnaire.

The writer desires to tender to each and all of those who have done him the honor to discuss the paper his hearty thanks for their courtesy and consideration.

He will take up each discussion in order and consider its salient points; and, finally, he will make a synopsis of all the important information obtained from the discussions, in addition to that accumulated through the questionnaire.

The fact pointed out by Mr. Metcalf, that creeping is caused by high rail ends even when the track is new, is both interesting and important, because it shows the necessity for extreme care in placing rails.

Mr. Metcalf's desire to see a more thorough discussion of the effect on creeping by the use of joints without slots for spiking has been gratified by Messrs. Disbrow, Keough, Blum, and Yardley. As they are all acknowledged experts on track, it is proper to draw the conclusion that splice plates of rails should not be slot-spiked to the ties, but that sliding of these plates on the timber or the tie-plates, as the case may be, should be permitted.

Mr. Metcalf's opinion that one should not attempt to anchor against creeping in both directions by means of the same tie is correct, and is shared by others as well as the writer.

The writer is not in accord with Mr. Metcalf's view that the determination of the resultant force of creeping is not worth while; because if ever an attempt is to be made really to check the creeping *in toto*, one must first know what stresses are developed when the motion is stopped.

Mr. Metcalf strikes a keynote when he says that "the greatest trouble to be met and overcome is misuse and even abuse of various types of rail-anchors." The number of such anchors should be properly adjusted to meet the tendency of the rails to creep. The general evidence shows that seldom are a sufficient number of anchors used to check the creeping effectively.

The writer is gratified by Mr. Metcalf's statement to the effect that "more intelligent study of local conditions to be overcome, and full co-operation on the part of all concerned properly to provide for them, would soon bring results so satisfactory that few would question the means to be used to meet any given condition." This, in reality, endorses the writer's suggestion that a systematic study of the entire matter be formally undertaken by the American Railway Engineering Association.

Mr. Disbrow's confirmation of the writer's stand taken more than four decades ago concerning the ultra-importance of a thorough system of drainage of both roadbed and right of way, is most gratifying, because it has been almost universally ignored on all except the really first-class roads, and even these have not given the matter proper attention until late years.

The stand taken by Mr. Disbrow against using rail joints as anti-creepers is sound and logical, and his advice on this point should be heeded by those in charge of track.

Answering Mr. Purdon's criticism of the suggested drastic treatment for excessive creeping on cheap track, the writer would state that he is convinced by the discussion as a whole that the "deadmen" would be too expensive and too troublesome to maintain; but he still thinks that the alternative scheme of driving a pair of square piles outside the rails in front of a large sawn tie would work satisfactorily, if an effective expansion arrangement was placed a short distance behind the anchorage. This would cause all track to be in tension, and, therefore, buckling would be impossible.

Mr. Jordan makes the statement that,

"Those who endeavor to belittle the importance of the subject [rail creeping] should be reminded that they are merely calling attention to their lack of contact with this extremely perplexing problem. Although it is true that rail creeping causes very little trouble under certain conditions of heavy track and light traffic, there are innumerable cases where the combating of this difficulty is a job for a Job."

The consensus of opinion of those answering the questionnaire is in accord with Mr. Jordan's claim, although some rather belittling remarks were offered about the importance of the subject treated. According to the expressed opinion of some experienced trackmen, rail creeping is the fundamental cause of almost all the ills that railroad operation is heir to; hence, due attention should be paid to Mr. Jordan's expert opinion on this matter.

Mr. Jordan makes an important addition to general knowledge when he remarks that "when steel creeps up grade on a single-track line, due to unbalanced traffic, this tendency is far greater at the lower end of the grade than elsewhere." His explanation of the phenomenon is both logical and interesting.

Mr. Jordan's "question as to whether any one has observed whether a hard-pulling, up-grade engine with a slow train actually pulls the steel down grade while the train effect reduces or reverses the result", is respectfully referred to the American Railway Engineering Association for solution. The writer agrees with Mr. Jordan that "dependable data on this question might be enlightening."

Mr. Jordan's adverse criticism of staggered joints receives the writer's hearty endorsement. Such joints certainly cause an unpleasant rocking motion of the cars, and it is logical to conclude that such rocking would aggravate track troubles in general, and especially the slewing of ties by creeping steel. It is to be hoped that the old standard practice of using even joints will soon be resumed.

Mr. Jordan's solution of the creeping question through a "rigid attachment gained by bolting splice-bars to the rail at the center and by slot-spiking to the ties," appeals to the writer as being sound. It seems to have been tried more or less in a desultory manner and with satisfactory results; but an elaborate experiment covering a long stretch of track would be well worth while.

Mr. Jordan's scheme of checking ordinary creeping "by using even joints and special tie-plates extending over two or three ties at each side of the joint" is worthy of a thorough trial by the American Railway Engineering Association. These long tie-plates, however, in the writer's opinion, should be entirely independent of the joint because, as previously indicated, the splice-bars should not be

slot-spiked to the ties, while the long tie-plates have to be so attached. One such detail placed at the middle of each rail and spiked or bolted to three ties might be found to suffice.

The writer is well pleased to note that such a high authority on track as Mr. Wagner endorses in general the conclusions drawn from the answers to the questionnaire.

Mr. Green's dissertation on rail creeping and its causes, is logical and interesting; but the writer has to disagree with him when he suggests the maintenance of track gauge at 4 ft. 8½ in. on curves, because, where the curves are sharp, the rigid wheel bases of passing locomotives and cars will certainly spread the track by loosening the spikes. It is better to confine such widening to legitimate amounts by properly increasing the gauge at the outset.

Mr. Hamilton's method of checking creeping in boggy ground was effective, but costly; hence, it is not likely to be adopted generally in muskeg country. His method of overcoming creeping for ordinary conditions of ground is good, but also somewhat costly. However, there should be no hesitation about spending any reasonable amount of money in the elimination of rail creeping.

Mr. Snyder's claim for the necessity of a closer co-operation between motive-power engineers and track engineers, is sound and timely, and it is to be hoped that it will receive proper consideration. His surmise that the practicable limit of loads from both locomotives and cars is now nearly reached is correct, unless there is a radical change in the present type of railroad track.

Mr. Snyder is right when he states that "a point has been reached where the whole subject of track design and construction should be restudied, not only by the men who design and maintain the track and roadway, but in co-operation with the men who design and operate the equipment that moves over the track and roadway." It is to be hoped that this suggestion will be acted on in the manner proposed by Mr. Snyder.

The writer suggests that Mr. Turner keep in touch with the experimenting that is being done concerning the creeping of rails in subways and prepare a paper on the subject for presentation to the Society for the benefit of the Profession.

Mr. Bontecou says that "in general, the author's 'Conclusions' seem to lay rather too much emphasis on the risk and cost due to rail creeping." This statement was made in the oral discussion, and, in reply, the writer pointed out that, according to Mr. Bontecou's own confession, he has been out of railroad practice for twenty years, that the great increase in weights of locomotives and cars had occurred mainly during the last two decades, and that rail creeping did not become the intolerable nuisance it is until the loads grew too heavy for the rails. In view of the foregoing facts, Mr. Bontecou's expressed opinion on the non-importance of the subject at issue should not be allowed to militate against further systematic study of how best to eliminate rail creeping.

The method of overcoming creeping suggested by Mr. Bryant, namely, by means of short angle-bars bolted to the rails and spiked to the ties, is well worthy of a systematic trial.

Mr. Marburg's suggestion of preventing tie creeping by means of ties with slanting bottoms may be worthy of elaborate experiment in the future; but

engineers must first be able to prevent entirely the motion of the rail on the ties before it becomes necessary to worry over preventing them from moving in the ballast. The suggestion, however, is a good one to bear in mind, and the writer is pleased that Mr. Marburg has put it on record in the Society's annals.

Mr. Buel seems to doubt whether, as an economic proposition, it would pay to spend on the railroad systems a sufficient sum to prevent creeping, and the problem is worth investigating. However, the question is one involving the safety of human lives—an item difficult to evaluate. In the writer's opinion, the correction of the evil will be found to be economic, even when that important consideration is ignored. Wrecks are expensive because of delay to traffic, damage to rolling stock, destruction of freight, and injury of employees—in addition to possible disaster for the traveling public; therefore, any expenditure within the bounds of reason that would effectively check the creeping, should be deemed economic.

As for the triangular ties mentioned by Mr. Buel as having been tried by the late James J. Hill, F. Am. Soc. C. E., and proved successful, the writer is of the opinion that, in soft ground, with the usual quantity and character of ballast, the tendency to sink under heavy loads would militate greatly against their general adoption.

Mr. Wilson's suggestion that American engineers consider reversion to the "bull-head" or "dumb-bell" section for rails, used so generally in Europe, is not, at first thought, as drastic as one might imagine because, although the supports are costly, the rails themselves can be made lighter and can be reversed when worn. If such rails would materially reduce the creeping, their adoption might ultimately prove to be economical. By importing a few miles of "dumb-bell" rails from England, with the chairs for their support, and placing them on a section of one of the American trunk lines where the creeping is pronounced and troublesome, an experiment could be made in a few years, which would determine the comparative economics of the English and the American types of rail section. Mr. Wilson's frankness in pointing out the fact that for the last three years the T-rail is replacing the "dumb-bell" rail in Continental Europe, is commendable; nevertheless, too much importance must not be placed on this circumstance, for the reason that during and after the World War the railroad men of France received visual proof of the economics of the American system of building railroads and therefore may be prone to adopt some features of our practice that are not as scientific or satisfactory as those which for many years have been in vogue in their own country. An example of this is their recent move to abandon the use of the inclined rail—a type of construction which is not only truly scientific but also conducive to longevity of rail and smoothness of track. The writer has felt so strongly opposed to this swing of the pendulum in France that he has lately sent to *Le Génie Civil* a word of warning to French engineers against dropping their scientific and satisfactorily proven practice and adopting an unscientific one in the hope of its being cheap in operation. In that communication he called attention to the fact that the use of the inclined rail is steadily on the increase in the United States, because whenever it has been tried in this country, as far as the writer can ascertain, it has given general satisfaction and has proved to be a money-saver.

The writer would be interested in learning whether Mr. Keith's statement under Class *a* is based on actual experience or on *a priori* reasoning. He imagines that it is the latter and, therefore, may be incorrect, for information has been furnished the writer by a high official of a Mexican railway to the effect that on a stretch of track lying idle for two years there was no rail creeping whatsoever. It would probably require an unusually heavy grade to induce rail creeping by gravity alone.

Dr. Lundie has stated that he would like to know of any method that can be suggested for attaching the rails to the ties. Angle-bars riveted to the rail, long enough to cover two, or better three, adjacent ties, and either screw-bolted or spiked to the latter, would make a very firm connection between rails and ties. If three of the ties were sawn and placed closely together, the angle-bars could be attached to all three, thus forming an exceedingly effective anchorage at mid-length of the rail.

The writer shares with Dr. Lundie the regret that "not a single engineer interviewed has presented facts which would enable one to compute the amount of potential creeping which should be provided for on a new railway line." Although this statement is strictly true, it must be remembered that Answer 15-*D* describes a fairly successful attempt to measure the resultant force causing creeping under specific conditions, and that Answers 15-*A* and 15-*C* give data from which the amount of the resistance (but not necessarily its maximum value) can be surmised. It was concluded from the only scientifically made series of experiments on rail thrust that "a rail-anchor should show at least 12 000 lb. of resistance before sliding on the rail."

Dr. Lundie's masterly analysis of the question of how to prevent rail creeping should be read by all railroad men who have anything to do with track maintenance. In it he discusses three general types of rail-anchors, and gives preference to the one in which the lower portion of the anchor is buried in the ballast and is entirely independent of the ties. The writer is inclined to agree with him, provided the lower end of each anchor is either expanded sufficiently to offer ample bearing surface against the ballast or else is rigidly attached to a buried beam, preferably of steel or cast iron. The forces to be dealt with are so great that a small bearing area at the lower end of the anchor would push the ballast ahead and upward and thus permit of rail motion. The writer has never seen any such rail-anchors as those suggested by Dr. Lundie, and he believes that their use would be an innovation which might lead to valuable results.

Dr. Lundie's concluding remark is most pertinent, and it should be heeded by railroad men, namely,

> "The merit of any rail-anchor should be judged by its efficiency in preventing rail movement and not by its price or ease of application, as is too often urged on the plea of economy of a very doubtful nature."

The writer deeply appreciates Mr. Skinner's complimentary remarks concerning his paper and the manner of its presentation, especially as Mr. Skinner is such a high authority on all questions relating to technical literature.

Mr. Skinner's suggested "projection, lock, or anchorage" might possibly be all right for direct rail bearing on ties, but it would rule out entirely the use of tie-plates.

Mr. Howard's indication of failure of metal at bolt holes as the result of repeated stresses indicates that the metal had been stressed well beyond the elastic limit, and illustrates the magnitude of the resisting forces developed in track by heavy traffic with resultant rail creeping. This, however, should not militate, as suggested by Mr. Howard, against attempting to arrest all longitudinal motion of the rails. He says "it is hardly conceivable that a close approach to the total arrest of longitudinal strains would be attempted or considered feasible to accomplish." The feasibility would probably depend entirely on the number of bolts or rivets used for attaching the anchor-bars to the rails; and this number can be increased at will simply by lengthening the bars or by increasing their number.

Mr. Howard's closing words are only too true, *viz.*:

"One of the most striking features brought out by the series of questions which were submitted to the railroads is the dearth of accurate knowledge of what creeping actually consists."

It is believed by the writer that the discussions throw a little more light on the subject—not enough, however, to clear up all uncertainties. It is, therefore, to be hoped that, as previously suggested, the American Railway Engineering Association will undertake a study of rail creeping and carry it out in the thorough manner characteristic of the committee work of that organization.

Mr. Chamberlin states, concerning the conflicting testimony furnished by the answers to the questionnaire, that "if all the facts in both cases were investigated and studied by the same mind, the conclusions might not be so much at variance after all." This is probably true, but, unfortunately, it would be exceedingly difficult to obtain the services of a super-railroad engineer of wide experience, who could be induced to devote sufficient time and energy to a study that would harmonize all the conflicting opinions advanced in the replies.

Mr. Chamberlin brings out a lamentable fact when he says that "engineers are rarely able to do all they wish to, but are compelled to do something less with the funds allowed them." The writer learned this to his sorrow many years ago while conducting an important technical investigation; but he is living in the hope that he will soon be called on to conduct a similar one in which he will be given a free hand.

From one of Mr. Chamberlin's remarks it appears that not all the railway engineers condemn the writer's suggested "deadmen" as being impracticable, although it must be confessed that the general consensus of opinion is adverse thereto.

It is gratifying to note that Mr. Chamberlin recognizes the indifference of many railroad men to the evils resulting from rail creeping, and that he endorses an attempt "by well directed and persistent efforts to enlist their co-operation in handling trains so that brakes would not be applied more suddenly than is necessary."

The experiences in Mexico on rail creeping related by Mr. Havens are very interesting and instructive, especially those concerning the effects on track of great variations in temperature.

The experiments made by Mr. Reeves in the Argentine Republic tend to confirm the claims of a number of railroad men to the effect that, although it is

feasible to overcome most of the creeping by building first-class track and using an ample number of anti-creepers, it is not practicable to check it entirely.

On the other hand, Mr. Robinson says,

"It is the writer's opinion that if the avoidable causes of creeping rails are removed, namely, poor drainage, narrow roadbed, insufficient ballast, and poor construction, any track can be prevented from creeping by the proper application and maintenance of standard makes of rail-anchors."

It is evident that authorities differ on this point as well as on many others; hence the necessity for exhaustive experimentation and investigation concerning all matters of major importance in railroad building and operation.

On learning that Messrs. R. Emmet Keough, C. F. Yardley, and Bernard Blum are high authorities on all matters relating to track, the writer sent each of them an earnest appeal to "do his bit" for the Engineering Profession by discussing not only the original paper, but also all the discussions thereon which had been published to that time by the Society, and to tell all that he knew of value concerning the subject at issue. All three responded to the appeal, and their contributions form a fitting conclusion to the discussion of this much vexed question.

Mr. Keough finds it quite possible to anchor rail sufficiently well to carry it through its full life without any ill effects due to rail creeping. He advocates laying the rails closely at summits of grades, so as to give them a chance to move down to the sag point in each direction, leaving at that point ample room for expansion. The same expedient is to be adopted at each station, water tank, grade crossing, etc. He lays stress on the necessity for applying the anchors as quickly as possible after a rail is laid, and he points out what has been indicated by others, *viz.*, that there are places where, even though every tie is anchored to the rails, creeping will not be entirely stopped.

In concluding his excellent discussion, Mr. Keough says "The writer believes that when furnished the necessary rail, ballast, and appliances to correspond to the traffic handled, it should be possible, at least in the majority of cases, to wear out a rail before experiencing much difficulty through creeping."

Mr. Yardley, whose experience with track has been unusually extensive, remarks,

"That the rail should be given free movement and that the joints should not be used to check the creeping of rail. In practically all instances, the joint ties under such conditions are loosened in the ballast and damaged through displacement of joint spikes, thereby creating a weak condition under that part of the rail which is considered the weakest portion of the structure—the joint."

He points out that the angle-bar should not extend beyond the edge of the rail flange, thus permitting spiking through the ordinary tie-plate, using a larger spike, if necessary. With such an arrangement he advises that suitable and sufficient frictional rail-anchors be installed on the base of each rail, against the same tie in opposite rails, to correct the creeping. When one rail tends to creep faster than the other, he advises the use of more anchors for that rail. This method he considers to be "the most economic and satisfactory method of anchoring." The writer, however, is of the opinion that anchoring to only one end of a tie would cause it eventually to slew, even though the ties used for anchoring at both their ends would tend for a while to prevent such slewing.

Mr. Yardley, as well as Mr. Keough, dwells forcibly on the necessity for installing anchors the same day that the rail is laid, and in sufficient quantities to hold the rail.

Mr. Yardley gives three excellent and convincing reasons for not rolling rails in such a manner as to provide an anchor at intervals throughout the rail-base, the bearing face to be integral with the rail.

He makes also this forceful statement:

"Unquestionably the minimum creeping of rails will be found in track protected with good drainage, substantial ballast of sufficient depth, and a high girder rail commensurate with the tonnage handled, and this should be the aim of every engineer in charge of maintenance, to the extent of even a gradual betterment process dependent on funds available to obtain this result."

Like most of the other experts, Mr. Yardley disapproves of the writer's rather drastic suggestion to check bad creeping on cheap track by means of "deadmen."

He points out that in many cases an insufficient number of rail-anchors is used to meet the conditions causing creeping, and very properly advises that more attention and intelligent study by trackmen be given to this matter.

Mr. Yardley's experience in the use of "Dutchmen" is quite interesting. It is a makeshift expedient to be adopted only when it is found impracticable to stop the creeping.

Mr. Blum's discussion offers some unfavorable criticism of the paper, and the writer is truly pleased that such is the case, because adverse comment is necessary in any investigation, if one is to attain the ultimate possibility in discovery of facts.

The reason why the writer sent his questionnaire to the Federal Managers or Presidents of the railroads instead of directly to the engineers and trackmen is that he was of the opinion that he would receive a greater number of answers and more elaborate ones by so doing; and his judgment proved to be right. No circular request to engineers for information would bring in anything like 70% of replies—possibly not more than 10%, judging not only by long experience in the past, but also by the circularizing that was done in relation to the discussions of this paper. Engineers are busy men, and, consequently, they are reluctant to do extra work if it can be avoided. In the case of the questionnaire, the Federal Managers and Presidents requested from their officers the information as a matter of routine business—hence the large number of replies.

The contradictoriness of the answers received might have been anticipated from the knowledge that the conditions affecting creeping vary so greatly in different localities; moreover, many of the responses came from trackmen who are not engineers and whose experience has been limited to a comparatively narrow field.

Of course, the writer did not subscribe to the correctness of the replies; but he drew his deductions from them as a whole, and included these in his résumé as his temporary determination "pending the discussions of the paper," as was indicated in his "General Conclusions." He is ready to modify any of them on irrefutable evidence given in the discussions as to their incorrectness.

The writer felt that he ought to give opposing views in his collection of replies for publication; and in the particular case mentioned by Mr. Blum, he included

it in order to evoke some comment in the discussion. The only reason he could evolve for a heavy rail expanding less than a light one was that the duration of the application of the heat must have been short; and the only reason for greater expansion was its greater power to overcome frictional resistance.

Relative to the question of the seriousness of rail creeping, accidents in the past have certainly occurred from this cause, and unless special care is taken, others will occur in the future. On first-class roads which are well looked after the danger is mostly—perhaps entirely—eliminated; but, on smaller roads, with insufficient ballast and not enough trackmen for proper supervision, it continues to exist.

As for anchorage to either "deadmen" or piles, it is simply a question of whether the rails and their fastenings will be able to withstand the stresses developed in them through the tendency to creep. If not, the method will be inapplicable, but, otherwise, it might prove economical under certain conditions.

Mr. Blum agrees with several other high authorities on track that the angle-bars at the joints should not be spiked to the rails, and the writer is glad to endorse this claim in his résumé of the discussions.

Summarizing, the writer finds from the discussions the following information, in addition to that which he compiled in his résumé of the replies to the questionnaire:

A.—High rail ends, even on new track, cause creeping.

B.—It is the opinion of a number of track experts that the splice-bars should not be used as rail-anchors. In other words, they should not be slot-spiked to the ties, but should be free to slide on their bearings.

C.—One should not attempt to anchor against creeping in both directions by means of the same tie.

D.—More rail-anchors are required to withstand creeping than are generally used, even on the best of tracks.

E.—When steel creeps up grade on a single-track line, due to unbalanced traffic, this tendency is far greater at the lower end of the grade than elsewhere.

F.—Even joints are less conducive to creeping than staggered joints.

G.—It is possible that the creeping evil can be cured by means of a rigid attachment gained by bolting long angle-bars to the rail at the center of the panel and by slot-spiking to the ties. This method should be given a thorough trial.

H.—It is possible also that short angle-bars tightly bolted to the rail and spiked or screw-bolted to a number of ties per panel will check creeping. This method, too, should be given a fair trial.

I.—The adoption of the "bull-head" or "dumb-bell" rail used in Europe would aid in the elimination of creeping; and this method should be tested and its economics determined.

J.—It is not advisable to attempt to check creeping by rolling a "projection, lock, or anchorage" on the bases of T-rails.

K.—There are only three successful types of anti-creeper in common use, but this by no means indicates that still better ones cannot be devised.

L.—Whether it is practicable to stop rail creeping entirely by the present anti-creeping devices is still a mooted point.

M.—Whether rail creeping can be effectively checked through the use of spikes is also still a mooted point.

N.—Anti-creepers with their lower ends buried in the ballast and entirely independent of the ties have been proposed, and are worthy of a thorough, practical test.

O.—The merit of any rail-anchor should be judged by its efficiency in preventing rail movement and not by its price or ease of application.

P.—Rail-anchors should be attached to each rail as it is laid and not afterward at some more convenient time.

Q.—Angle-bars should not extend beyond the edge of the base of the rail, in order to permit spiking through the ordinary tie-plate.

R.—Although the "deadmen" suggested by the writer are disapproved as being cumbrous, troublesome, and expensive, his pile anchorage, placed in front of a group of closely spaced ties which are thoroughly anchored to the rails by long angle-bars, leaving a proper arrangement for rail expansion directly behind each anchored panel, with the anchorages spaced at such intervals as are found to be necessary by experience, would probably prove an effective device for inexpensively stopping excessive creeping on cheap track, especially when the motion is in one direction only. It would be well to give this suggested scheme a fair trial before passing on it an adverse judgment that would rule it out of use.

S.—Belittling of the evils due to rail creeping is unwarranted and is conducive to a postponement of their correction.

T.—There should be no hesitation about spending any reasonable amount of money in the elimination of rail creeping.

U.—Closer co-operation between the designers of track and the designers of rolling stock would be conducive to the elimination of some of the rail creeping.

V.—An urgent necessity exists for a further study, on an elaborate programme, of the phenomenon of rail creeping, its causes, and its elimination; and no organization is so well fitted to do this work as the American Railway Engineering Association.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1471

NOTES ON IMPACT

By F. W. Gardiner,* M. Am. Soc. C. E.

With Discussion by Messrs. George Paaswell, L. J. Mensch, F. E. Turneaure, and F. W. Gardiner.

Synopsis.

Numerous recent tests have shown that the dynamic effect of rolling loads on bridges has been much over-estimated, and this is particularly true of electric trains in which the loads are smooth rolling wheels and the mechanism has no reciprocating motion.

The investigation presented in the paper shows the relation between the static and dynamic stresses in girder spans for smooth rolling loads, and the following conclusions have been reached:

1.—For speeds that obtain in railroad practice, the dynamic effect of smooth rolling loads is so small that it may be neglected.

2.—There is a critical speed that produces a maximum dynamic increment of stress for a single load, and this speed is equal to $15\,184\,\frac{r}{L}$ ft. per sec., or $10\,329\,\frac{r}{L}$ miles per hour, where r is the radius of gyration and L the length, in inches.

3.—The dynamic increment of stress for a single load for the critical speed is 31.8 per cent.

4.—That even for the maximum speed condition, with the wheel spacing usual on electric trains, the combined dynamic effect of four wheels may be less than the static effect.

The following is an investigation of the stresses produced in a beam, supported at the ends, by a load rolling over it from one support to the other. The object of making the investigation is to determine the relation between the stresses produced by a smooth rolling load and those produced by a static load.

If the effect of the mass of the beam is omitted in the investigation, it will be found that the stress produced by a smooth rolling load is equal to the static stress plus a small increment due to the centrifugal force of the load moving in the arc formed by the elastic curve. The effect of centrifugal force is also omitted in this discussion.

* New York City.

This statement is not new and may be found in some recent textbooks on Mechanics. A short proof of this statement is given because it is necessary that it be made clear before proceeding to investigate the effect of the mass of the beam.

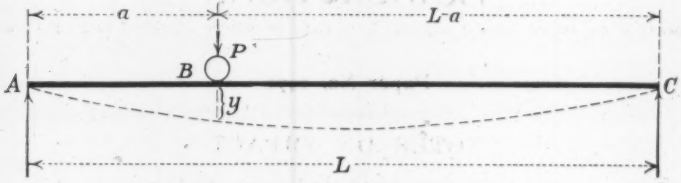

FIG. 1.

Let $A\,C$, Fig. 1, be a beam supported at A and C, and P a load rolled on from A to C. The deflection, y, produced by a static load, P, at the distance, a, from the support is

$$y = \frac{P}{6\,E\,I\,L}\left[2\,a^2\,(L-a)^2\right]$$

The internal work done on the beam by a load, P, rolling from A to B is

$$W_i = \frac{P^2\,a^2}{6\,E\,I\,L}\,(L-a)^2$$

The average force acting through the deflection, y, is equal to the work divided by the distance

$$\frac{W_i}{y} = \frac{P^2\,a^2}{6\,E\,I\,L}\,(L-a)^2 \times \frac{6\,E\,I\,L}{P\,[2\,a^2\,(L-a)^2]} = \frac{P}{2} \quad \ldots\ldots\ldots\ldots (1)$$

The whole load, P, has passed through a vertical distance, y, when the load has reached a position, B, distant a from the support, A, and, therefore, the total external work is Py. It has been shown that one-half of this work, $\frac{P}{2}y$, is absorbed by internal work in the beam. The unbalanced average load, $\frac{P}{2}$, acting on the mass of the load, P, and accelerating it down the incline formed by the elastic curve, will have stored energy in the acceleration of the mass of an amount, $\frac{P}{2}y$, when the load has reached Position B. The whole work of the load, P, is absorbed, therefore, by the static deflection and the acceleration of the mass of the load. It will be noted that the component of P producing acceleration of the mass is tangential to the elastic curve and the change of direction of this acceleration produces a centrifugal force radial to the curve.

It is true, therefore, as stated, that the stress produced by a smooth rolling load at any velocity is the static stress plus a small increment due to centrifugal force, when the effect of the mass of the beam is not considered.

It is now proposed to consider the effect of the mass of the beam.

Let $A\,C$, Fig. 2, represent a beam supported at A and C, and P a load rolling from A to C with a constant velocity, V.

It is proposed to write the equation of the distance-time curve of the deflection of the center of the beam.

Let m be an equivalent mass concentrated at the center of gravity of the mass of the beam, and assume this equivalent mass to be constant during the deflection. The relation of this mass to the total mass of the beam will be shown later.

Let A = the vertical acceleration of the mass, m;
y = the vertical deflection of the center of the beam;
t = the time the load, P, takes to go a distance, a.

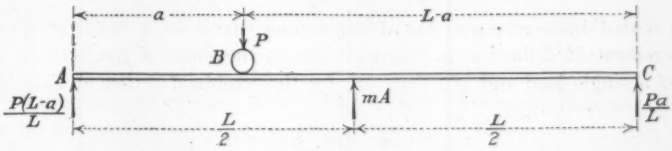

FIG. 2.

The deflection of the center of the beam for any position of the load, P, will be the static deflection minus the deflection due to the force, $m\,A$, applied at the center of the beam.

The equation of the distance-time curve of the deflection of the center of the beam is, therefore, for all values of a from 0 to $\frac{L}{2}$:

$$y = \frac{Pa}{48\,E\,I}(3\,L^2 - 4\,a^2) - \frac{m\,L^3}{48\,E\,I}\frac{d^2\,y}{d\,t^2} \quad \ldots\ldots(2)$$

Let $\frac{P}{48\,E\,I} = C_1$; $\frac{m\,L^3}{48\,E\,I} = C_2$; $a = C_3\,t$, where C_3 = velocity.

Then $$y = C_1\,C_3\,t\,(3\,L^2 - 4\,C_3^2\,t^2) - C_2\frac{d^2\,y}{d\,t^2} \quad \ldots\ldots(3)$$

$$y + C_2\frac{d^2\,y}{d\,t^2} = C_1\,C_3\,t\,(3\,L^2 - 4\,C_3^2\,t^2) \quad \ldots\ldots(4)$$

The solution of this differential equation gives

$$y = -3\sqrt{C_2}\,C_1\,C_3\,L^2 \sin\frac{t}{\sqrt{C_2}} + 3\,C_1\,C_3\,L^2\,t - 4\,C_1\,C_3^3\,t^3 \quad \ldots\ldots(5)$$

and this is the equation of the distance-time curve of the deflection of the center of the span for values of t from zero to a value making the distance traveled by the load one-half the span length. It will be noted that in this equation the first term is the effect of the mass of the beam, and the last two terms the elastic deflection in a beam of no mass. The equation for the remaining half of the beam has the same first term and a remaining term giving symmetrical values about the center line with those given in Equation (5). This equation is, therefore, all that is necessary to plot the distance-time curve. The detail of the solution of the differential equation is given in Appendix A.

The value to be given to the equivalent mass, m, acting at the center of gravity of the beam was found by plotting the elastic curve for ten positions of the load between the support and the center of the beam and computing the vertical displacement of the center of gravity of the mass. It was found that

for a deflection, y, at the center, the center of gravity of the mass was deflected a little less than $\frac{2}{3}\,y$, and that this relation was nearly constant throughout the whole deflection. A value of two-thirds of the total mass of the beam has been used, therefore, for the quantity, m, in the equation.

Graph No. 1, Fig. 3, is a graph of this equation for a rather flexible girder, 39 in. deep and of 51-ft. span, with four wheels of 12 500 lb. each passing over it at a speed of 120 ft. per sec., or 82 miles per hour, representing a high railroad speed.

The dotted lines represent the distance-time curves of a beam of no mass. The increment of deflection in this case due to the mass of the beam is about 10% for a single load and less than 3% for the combined action of four wheels.

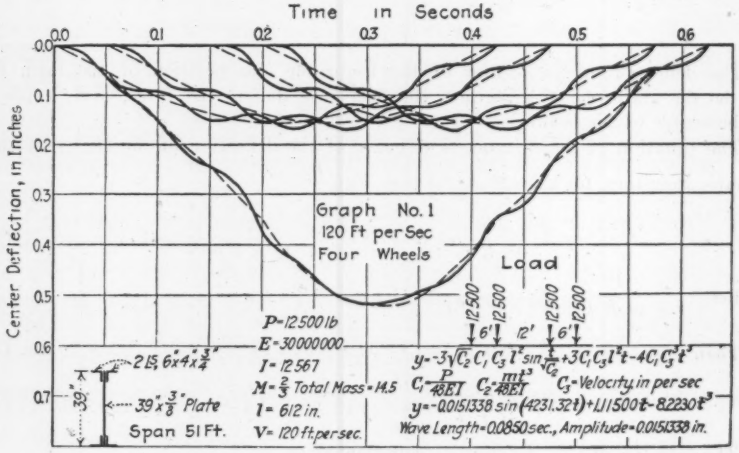

FIG. 3.

A graph of the same girder with the same load passing over it at a speed of 30 ft. per sec., or 20.4 miles per hour, would show that the wave motion caused by the mass inertia has the same time length and one-quarter the amplitude of the first graph, and as the velocity is decreased the wave amplitude will decrease and the graph approach closer and closer to the curve of no mass.

Graph No. 2, Fig. 4, is a graph of the same girder with the same load passing over it at a speed of 400 ft. per sec., or 273 miles per hour. The wave in this case has the same time length and $3\frac{1}{3}$ times the amplitude of Graph No. 1. The increment of deflection due to a single load is in this case 31.8% and this is the maximum increment that can be produced by a single load, as will be shown later. For speeds greater than 400 ft. per sec., the value of the deflection decreases. Graph No. 3, Fig. 4, is a graph for the same girder with the same load passing over it at a speed of 1 200 ft. per sec., or 830 miles per hour. The deflection in this case is about 6% of the no-mass deflection and

as the speed increases the deflection will decrease, approaching the value zero when the speed becomes infinite.

A study of these graphs makes it evident that there is a speed which produced a maximum mass effect for a single load, and an examination of the equation enables this value to be found. Consider the first term of the equation which, as previously stated, gives the mass effect, that is,

$$-3\sqrt{C_2}\,C_1\,C_3\,L^2 \sin\frac{t}{\sqrt{C_2}}$$

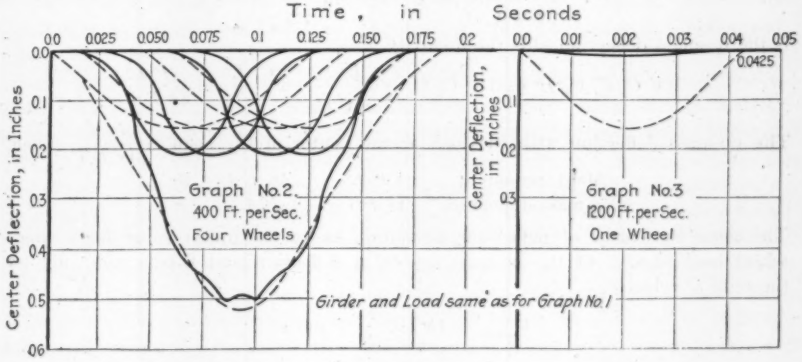

FIG. 4.

The inertia of the mass produces a maximum effect when the velocity has a value such that the wheel load arrives at the center of the span in a time, t, having a value such that the angle, $\frac{t}{\sqrt{C_2}} \times \frac{180}{\pi}$ is 270°, and, therefore, the sin is -1. Let this value of the velocity be V. Then,

$$\frac{t}{\sqrt{C_2}} \times \frac{180}{\pi} = 270, \text{ or } t = \frac{3\,\pi\,\sqrt{C_2}}{2}$$

$$V = \frac{L}{2\,t} = \frac{L}{2} \times \frac{2}{3\,\pi\,\sqrt{C_2}} = \frac{L}{3\,\pi\,\sqrt{C_2}} = \frac{L}{3\,\pi}\sqrt{\frac{48\,E\,I}{m\,L^3}}$$

$$V^2 = \frac{L^2}{9\,\pi^2} \times \frac{48\,E\,I}{m\,L^3} = \frac{16\,E\,I}{3\,\pi^2\,m\,L}$$

Since

$$m = \frac{A\,L\,w}{g} \times \frac{2}{3},$$

$$V^2 = \frac{16\,E\,I}{3\,\pi^2\,L} \times \frac{3\,g}{2\,A\,L\,w} = \frac{16\,E\,A\,r^2}{\pi^2\,L} \times \frac{g}{2\,A\,L\,w} = \frac{16\,E\,g\,r^2}{2\,\pi^2\,L^2\,w}$$

$$V = \frac{4\,r}{\pi\,L}\sqrt{\frac{E\,g}{2\,w}} = \frac{r}{L} \times \frac{4}{\pi}\sqrt{\frac{E\,g}{2\,w}} = \frac{r}{L} \times \text{(a constant)}.$$

For a steel girder the value of this constant is

$$\frac{4}{3.1416}\sqrt{\frac{30\,000\,000 \times 32.2 \times 12}{2 \times 0.283}} = 182\,213$$

and the value of the velocity giving the maximum mass effect is

$$V_{max.} = 182\,213\,\frac{r}{L} \text{ in. per sec., or } 15\,184\,\frac{r}{L} \text{ ft. per sec.}$$

in which r is the radius of gyration and L the length, in inches.

The deflection for velocity, V, or when

$$V = \frac{L}{3\,\pi\,\sqrt{C_2}} = C_3, \text{ and } \sin\frac{t}{\sqrt{C_2}} = -1$$

is the no-mass deflection plus the quantity

$$3\,\sqrt{C_2}\,C_1\,C_3\,L^2 = 3\,\sqrt{C_2}\,C_1\,\frac{L^3}{3\,\pi\,\sqrt{C_2}} = \frac{C_1\,L^3}{\pi} = \frac{P\,L^3}{48\,E\,I\,\pi}$$

The no-mass deflection with the load at the center of the span is $\frac{P\,L^3}{48\,E\,I}$. Then,

$$\frac{\text{Mass increment}}{\text{No-mass deflection}} = \frac{P\,L^3}{48\,E\,I\,\pi} \times \frac{48\,E\,I}{P\,L^3} = \frac{1}{\pi}$$

The mass increment of deflection, therefore, has a maximum value for a single wheel load 31.83% of the no-mass or static deflection and this occurs only for the critical velocity,

$$V_{max.} = 15\,184\,\frac{r}{L} \text{ ft. per sec.}$$

The girder for which the graphs are shown has a value of $\frac{r}{L} = 0.0263$ and a value of $V_{max.} = 400$ ft. per sec., or 271 miles per hour. The graph, Fig. 4, is, therefore, the maximum condition for a single load.

An average value for new elevated railway structures is $\frac{r}{L} = 0.0397$, for which $V_{max.} = 600$ ft. per sec., or 410 miles per hour.

The stress in any girder is a constant times the deflection, and the relation between the deflection at the center, and the no-mass or static deflection given by Equation (5) is the relation sought to be established by this investigation.

Test to Determine the Deflection of Girders Due to a Moving Load.

A test was made on February 17th, 1918, on track stringers on the express track of the Third Avenue Line of the Manhattan Railway Division of the Interborough Rapid Transit Company, New York City. Two composite cars were loaded with brake shoes to equal the live load for a fully loaded train. The test was made in Span 806 at 91st Street. The track stringers in this span are plate girders, 45 ft. long and 39 in. deep, with ½-in. web spliced at the mid-point, and each flange consisting of two 6 by 6 by ⅝-in. angles.

Two independent methods were used simultaneously to ascertain the deflections, as follows:

One method consisted of fastening a piano wire to the end stiffeners of the track girder, stretched tightly along the neutral axis of the stringer and distant

about 4 in. from its face. At the middle of the span a board was wedged tightly between the top and bottom flanges. On the side of the board facing the wire, and directly back of it, a card was fastened with thumb tacks to hold it in place. A pencil firmly fastened to the wire with its point in contact with the paper registered any vertical movement of the girder. After each test the card was removed, properly labeled for identification, and a new card put in place. Later, the length of line scribed on the card was scaled. The length so measured represented the true deflection of the girder due to a particular load at a definite speed.

The second method was by direct rod readings, using sliding level rods provided with strap iron hooks so fastened to the upper section of the rod that it could be suspended from the flange of the girder by allowing the point of the hook to rest on the top surface of the horizontal leg of the lower flange angle. With the rod free to slide and the bottom of the lower section resting on the street surface below, the movement between the free sections of the rod will indicate the vertical deflection caused by any live load passing over the girder. At a convenient place opposite the sliding sections of the level rod, a horizontal strip of metal was fastened on the side of the lower section or fixed part of the level rod and extended beyond the side of the upper section or movable part. Back of the strip of metal a card was fastened on the upper or movable section. With the rod hanging from the girder, with the parts free to slide, and the bottom in contact with the street below, the rod is in position to register deflections. For each reading three rods were used, one at each end and one at the middle. When the rods were so placed, a pencil line was drawn across the top edge of the metal strip on the card and the point of the pencil was kept in contact with the card. As the load passed over the girder, the mark made by the pencil recorded the true deflection of the girder due to the live load, at the center and ends. The rod readings were taken at the ends and the center simultaneously to eliminate deflections of other parts of the structure, and the recorded deflection at the center was corrected accordingly. As with the first method, the deflections were obtained by scaling the length of line scribed on the card. The two methods gave results which in no case differed more than $\frac{1}{32}$ in.

The test was begun by first placing the cars on the girders in the position to give maximum deflection for static live load.

The train was then run over the track girder at various speeds and the deflection noted. The speed was measured by placing markers along the track 176 ft. apart, or 88 ft. each way from the center of the girder. The train was then run at various speeds. By means of a registering device on the train, the time required to pass over a distance of 176 ft. was recorded, and read to tenths of a second, and from this information the speed in miles per hour was computed. Ten trips were made, with speeds varying from $7\frac{1}{2}$ to $33\frac{1}{3}$ miles per hour. The results of these tests are shown on Fig. 5. Some of these results indicate that:

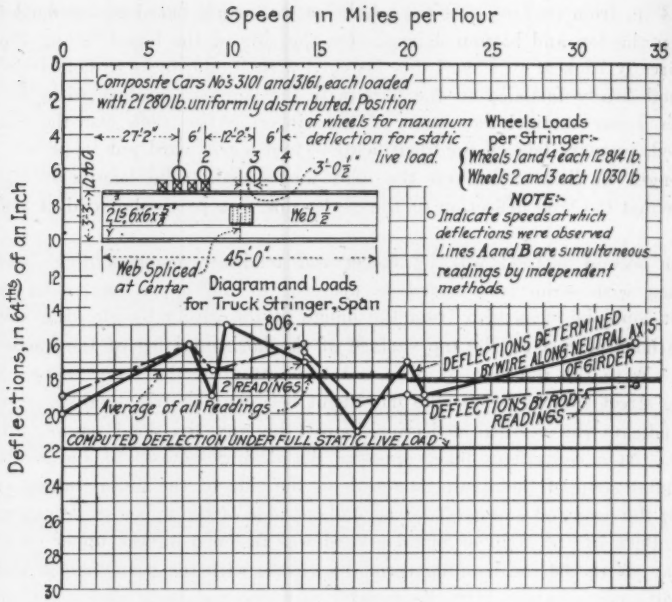

FIG. 5.

1.—The computed deflection for static load is greater than the measured deflection caused by the static load.

2.—The observed deflections for moving load were less than the observed deflection for static load, except at a speed of 17 miles per hour, in which case the deflection was slightly greater.

APPENDIX A.

The solution of the differential equation:

$$y + C_2 \frac{d^2 y}{d t^2} = C_1 C_3 t (3 L^2 - 4 C_3^2 t^2) \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

is as follows:

The units in this equation must be consistent: y is deflection, in inches; the expression, $\frac{d^2 y}{d t^2}$, is acceleration, in inches per second, and therefore the value of m in C_2 is weight, in pounds, divided by acceleration of gravity, in inches per second; the velocity, C_3, is in inches per second; the span length is in inches and time, t, is in seconds. A particular solution of the equation for mass, $m = 0$, therefore, $C_2 = 0$, gives

$$y = C_1 C_3 t (3 L^2 - 4 C_3^2 t^2) \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

This value of y, plus the value of y found on the hypothesis that the second member is zero, will be the complete value of y.

$$y + C_2 \frac{d^2 y}{d t^2} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Let $\qquad y = e^{mt}; \quad \frac{d y}{d t} = m e^{mt}; \text{ and } \frac{d^2 y}{d t^2} = m^2 e^{mt}.$

Substitute in Equation (7):

$$e^{mt} + C_2 m^2 e^{mt} = 0$$
$$1 + C_2 m^2 = 0$$

$$m = \frac{1}{\sqrt{C_2}} \left(\pm \sqrt{-1} \right), \text{ or } \left(0 + \frac{1}{\sqrt{C_2}} \sqrt{-1} \right) \text{ and } \left(0 - \frac{1}{\sqrt{C_2}} \sqrt{-1} \right)$$

For each pair of imaginary values of m, or $\left(a + b\sqrt{-1}\right)$ and $\left(a - b\sqrt{-1}\right)$, there will be a term, $A e^{at} \cos b t + B e^{at} \sin b t$, A and B being arbitrary constants. Then

$$y = A \cos \frac{1}{\sqrt{C_2}} t + B \sin \frac{1}{\sqrt{C_2}} t.$$

For $y = 0$, $t = 0$, therefore $A = 0$, and

$$y = B \sin \frac{1}{\sqrt{C_2}} t = B \sin \frac{1}{\sqrt{C_2}} t + C_1 C_3 t \left(3 L^2 - 4 C_3^2 t^2 \right)$$

$$= B \sin \frac{1}{\sqrt{C_2}} t + 3 L^2 C_1 C_3 t - 4 C_1 C_3^3 t^3.$$

$$\frac{d y}{d t} = B \cos \frac{t}{\sqrt{C_2}} \times \frac{1}{\sqrt{C_2}} + 3 L^2 C_1 C_3 - 12 C_1 C_3^3 t^3.$$

But $\frac{d y}{d t} = 0$ for $t = 0$, hence

$$0 = \frac{B}{\sqrt{C_2}} + 3 L^2 C_1 C_3, \text{ or } B = -3 L^2 \sqrt{C_2} C_1 C_3.$$

$$y = -3 \sqrt{C_2} C_1 C_3 L^2 \sin \frac{t}{\sqrt{C_2}} + 3 C_1 C_3 L^2 t - 4 C_1 C_3^3 t^3 \ldots\ldots (5)$$

DISCUSSION

GEORGE PAASWELL,* M. AM. SOC. C. E. (by letter).—The subject of impact is somewhat hazy, and, in attempting its rational analysis, Mr. Gardiner is entitled to much appreciation. Impact, in the sense that it is used in the paper, is probably a misnomer. More properly defined, it is the dynamic effect of a rolling load that is sought. In the ordinary Newtonian sense, impact is a collision implying suddenness of application with double to triple the static effect of simple pressure. Transverse impact, where there is sudden application of load, is itself a separate and important problem.

The dynamic effect of rolling loads, the subject of the paper, has been completely analyzed, and those interested should read "The Annotated Clebsch" by St. Venant, Note du No. 61. A synopsis and detailed discussion of this note are given by Karl Pearson in "The Elastical Researches of Barre de St. Venant", pp. 230 *et seq.* The problem of the rolling load received its first detailed study at the hands of Professor Willis and is known as the Willis problem. Its study was occasioned by the classic report on the use of steel (1849), made by the Iron Commissioners. The solution, in which the British mathematical physicist, Stokes, collaborated, is based on a particle rolling along a bar, with the inertia effects of the bar ignored. This latter was a serious omission and, as experiments demonstrated, vitiated the formulas derived by Stokes. Phillips and St. Venant completely analyzed this problem, taking all factors into consideration, and, although the analysis is by no means simple, the formulas are of a workable nature. St. Venant analyzed not only this dynamic effect of the rolling load, but also the effect of a transverse and longitudinal impact on a beam. For the case of a body falling on a beam or striking it, as, for example, the locomotive counterweight, this analysis is of importance.

There are really two problems in actual practice: (1) The dynamic effect of a rolling load—which is the subject of the paper; and (2), the effect of the impact due to track and rolling stock irregularities. It is the second problem, which, coloring the solution of the first, makes formulative analysis of questionable value. It is true, however, that the rolling effects of electric trains tend to subordinate the importance of the second problem, and Mr. Gardiner's analysis, properly carried out, may lead to rational impact factors.

In analyzing the rolling load, St. Venant has pointed out that maximum deflections do not necessarily imply maximum stresses, and it is rather to maximum curvatures that one must look for controlling stresses. Such curvatures may be obtained from the time-deflection curve by substituting the value of $x = V\,i$. Multiplying the curvature by the rigidity factor, $E\,I$, gives the bending moment.

The solution, Equation (5), of the differential Equation (4) is not strictly correct, as is readily noted by substitution back. The exact form of the solution is:

$$y = 3\,L^2\left(1 + \frac{m\,V^2\,L}{6\,E\,I}\right)\left(-\,C_1\sqrt{C_2}\,C_3\sin\frac{t}{\sqrt{C_2}} + C_1\,C_3\,t\right) - 4\,C_1\,C_3^3\,t^3$$

However, the correction, $\dfrac{m\,V^2\,L}{6\,E\,I}$, is negligible for actual velocities and the expression given by Mr. Gardiner is sufficiently exact for all purposes.

* New York City.

To determine the value of the equivalent mass factor, the writer has not attempted to check the graphical determination of $\frac{2}{3}$ obtained for this factor, but St. Venant has given the general expression for the mass coefficient, k, in the following form:

$$k = \int_0^a \left(\frac{u}{u_a}\right)^2 \frac{d\,z}{a}$$

where u is the deflection at any point, z, from the terminal due to a load at a, and u_a is the deflection at the point, a. The value for a freely supported beam is given by:

$$k = \frac{1}{105}\left[1 + 2\left(\frac{a}{L-a} + \frac{L-a}{a} + 3\right)^2\right]$$

An average value taken over the distance, $\frac{L}{2}$, gives about 0.7, which agrees closely with the value given.

To determine the conditions for maximum deflection it must be noted that the sine function becomes equal to -1 for a series of values of

$$\frac{t}{\sqrt{C_2}} = \frac{4n+3}{2}\pi$$

where n takes all the integral values from zero on. Correspondingly, there is a series of maximum deflections for the various values of n, decreasing in value as n increases. The series of critical velocities now become:

$$V = \frac{L}{(4n+3)\,\pi\sqrt{C_2}}$$

and the deflection increases are

$$\frac{3}{\pi(4n+3)}$$

Table 1 shows the relation between the several values of n, the critical velocity, and the excess of the dynamic deflection over the static.

TABLE 1.

n.	Critical velocity, in miles per hour.	Percentage of increase.
0	270	31.8
1	117	13.6
2	74	8.7
3	54	6.5
10	19	2.5

It must be noted again that these dynamic excesses cover merely deflections, and that maximum curvatures must also be analyzed to determine the proper stresses developed.

In examining the results of the experiments, the fact that the static deflection as observed was less than that computed may be due to the terminal conditions. In the usual elevated railroad design the ends are practically fixed, or where there is an expansion bearing, one end is fixed and the other free. In the former case, the

deflection is one-quarter, in the latter, one-half of the deflection of a free terminal girder.

To conclude, the analytical and experimental results of the paper are such that the subject of impact due to rolling loads must be thoroughly investigated, and if there has been wasteful design due to excessive allowances, future construction must not repeat such extravagances.

L. J. Mensch,* M. Am. Soc. C. E. (by letter).—The vibrations of a beam under a moving concentrated load were first investigated by Dr. H. Zimmermann.† Curiously enough Dr. Zimmermann did not consider the influence of the inertia of the beam, and confined his investigations to the effect of the centrifugal force.

Mr. Gardiner and his unnamed original investigators considered only the effect of the inertia of the beam and neglected the effect of the centrifugal force. Their conclusions must be wrong, as there is a serious error in the solution of the differential Equation (4). For, by successively differentiating Equation (5), there is obtained:

$$\frac{d\,y}{d\,t} = -3\sqrt{C_2}\,C_1\,C_3\,L^2\,\frac{1}{\sqrt{C_2}}\cos\frac{t}{\sqrt{C_2}} + 3\,C_1\,C_3\,L^2 - 12\,C_1\,C_3^3\,t^2$$

$$\frac{d^2\,y}{d\,t^2} = 3\sqrt{C_2}\,C_1\,C_3\,L^2\,\frac{1}{C_2}\sin\frac{t}{\sqrt{C_2}} - 24\,C_1\,C_3^3\,t$$

$$C_2\frac{d^2\,y}{d\,t^2} = 3\sqrt{C_2}\,C_1\,C_3\,L^2\sin\frac{t}{\sqrt{C_2}} - 24\,C_1\,C_2\,C_3^3\,t$$

By adding this to the value of y given by Equation (5):

$$y + C_2\frac{d^2\,y}{d\,t^2} = -24\,C_1\,C_2\,C_3^3\,t + C_1\,C_3\,t\,(3\,L^2 - 4\,C_3^2\,t^2)$$

which does not agree with Equation (4). Moreover, a simple consideration shows that the inertia of a beam should always have the effect of decreasing its deflection.

Both the author and Dr. Zimmermann agree that the greatest effect of a moving load occurs at a section near the center of the beam, and, in investigating only that section, the whole problem can be solved without the aid of calculus.

Let r = radius of curvature of the beam in the center;
y_1 = deflection in the center under a static load at the center;
y = " " " " " a moving load when it arrives at the center;
m = mass of the beam and floor construction;
m_1 = " " " moving concentrated load;
p = acceleration produced by a force other than gravity acting on a body;
t = time it takes for the load to go the distance, $\frac{L}{2}$;
P = concentrated moving load at the center;
C = centrifugal force acting downward when the load is at the center;
F = force it takes to accelerate the beam downward in the time, t;
M = moment produced at the center of the beam;

* Chicago, Ill.

† "Die Schwingungen eines Trägers mit bewegter Last," Berlin, 1896.

It is well known that

$$\frac{1}{r} = \frac{M}{E\,I} \quad \ldots\ldots (8)$$

$$y_1 = \frac{P\,L^3}{48\,E\,I}$$

and, similarly,

$$y = (P + C - F)\,\frac{L^3}{48\,E\,I} \quad \ldots\ldots (9)$$

$$M = (P + C - F)\,\frac{L}{4} \quad \ldots\ldots (10)$$

From Equations (8), (9), and (10),

$$y = \frac{1}{r} \times \frac{L^2}{12} \quad \ldots\ldots (11)$$

From Mechanics,

$$C = m_1\,\frac{v^2}{r} \quad \ldots\ldots (12)$$

and by introducing $\frac{1}{r}$ from Equation (11),

$$C = \frac{12\,m_1\,v^2\,y}{L^2} \quad \ldots\ldots (13)$$

From Mechanics, it is also known that force, F, equals mass times acceleration, and it has been known for a long time that the ideal mass, concentrated at the center, is five-eighths of the mass of the entire beam; therefore,

$$F = \frac{5}{8}\,m \times p \quad \ldots\ldots (14)$$

Now, distance equals half the acceleration times the square of the time, or

$$y = \frac{p}{2}\,t^2 \quad \ldots\ldots (15)$$

or,

$$p = \frac{2\,y}{t^2} \quad \ldots\ldots (16)$$

also,

$$\frac{L}{2} = v \times t \quad \ldots\ldots (17)$$

Introduce the value of t from Equation (17) into Equation (16),

$$P = 8\,v^2 \times \frac{y}{L^2}$$

and,

$$F = \frac{5}{8}\,m \times \frac{8\,v^2\,y}{L^2} = \frac{5\,m \times v^2 \times y}{L^2} \quad \ldots\ldots (18)$$

From Equations (10), (13), and (18),

$$y = \frac{\left(P + \frac{12\, m_1\, v^2\, y}{L^2} - \frac{5\, m\, v^2\, y}{L^2}\right) L^3}{48\, E\, I}$$

or,

$$y = \frac{P\, L^3}{48\, E\, I \left(1 + \frac{5\, m\, v^2\, L}{48\, E\, I} - \frac{m_1\, v^2\, L}{4\, E\, I}\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

$$y = \frac{P\, L^3}{48\, E\, I} \times \frac{1}{1 - \frac{m_1\, v^2\, L}{4\, E\, I}\left(1 - \frac{5}{12}\frac{m}{m_1}\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

$$y_1 = \frac{P\, L^3}{48\, E\, I}$$

and,

$$\frac{y}{y_1} = \frac{1}{1 - \frac{m_1\, v^2\, L}{4\, E\, I}\left(1 - \frac{5}{12}\frac{m}{m_1}\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

All dimensions should be in inches, and masses should equal loads, in pounds, divided by acceleration of gravity, in inches per second; v also should be in inches per second.

No dynamic effect occurs when $m_1 = \frac{5\, m}{12}$. When m_1 is less, the dynamic effect is less than the static effect. As a rule, the dynamic effect is very small, reaching in rare cases, especially in reinforced concrete structures, a value 10% higher than the static effect.

In a similar simple way the effect of a system of loads can be treated, and the influence of the deflection of trusses and arches, supporting the longitudinal girders, can be studied. The effect of the centrifugal force can easily be nullified by giving the structures a small crown at the center, which, by experienced designers, is nearly always done.

F. E. Turneaure,* M. Am. Soc. C. E. (by letter).—The author's paper brings out some interesting and useful relations concerning the effect of rapidly moving loads on short beams. The most valuable part is his general conclusion, implied rather than stated, that the increased stresses due to the effect of speed alone are of no consequence at speeds used in practice. Although the writer agrees fully with this general conclusion, there are some parts of the paper which he would criticize.

The theoretical analysis appears to be divided into two distinct parts, the first being intended to prove that the stress produced in a beam by a rolling load is equal to the static stress plus a small increment due to the centrifugal force of the load moving in a curved path. In the first place, it does not seem that this statement needs any proof, and, in the second place, it appears to the writer that the analytical work presented by the author does not prove this at all, but relates to something wholly different.

* Madison, Wis.

Mr. Gardiner first shows that, considering static deflection only, the energy absorbed by the beam during its deflection to its maximum point is only half of the total energy given up by the body in dropping a distance equal to this deflection. The other half of this available energy must be used in producing tangential acceleration of the body as it rolls down the sloping path. Up to this point the statements seem to be correct, but the author then goes on to say that this tangential acceleration of the body produces a centrifugal force radial to the curve, or, to quote the paper, "the change of direction of this acceleration produces a centrifugal force radial to the curve." As a matter of fact, a force is produced or a pressure developed, not by a change in direction of acceleration, but by a change in velocity, that is, by an acceleration. Furthermore, the centrifugal force radial to the curve is not caused by a tangential change in velocity, but by a radial change in the velocity, or by the radial acceleration. Computations† will show that the tangential change in velocity due to the absorption of the small amount of energy mentioned is exceedingly minute, amounting, for example, to, perhaps, 0.006 of 1% of the original velocity where that velocity was 60 miles per hour and the span length 50 ft.; but the radial change in velocity, or the radial acceleration, is a considerable quantity, as it is a function not of the change in tangential velocity, but of the actual tangential velocity itself, and the greater this is, the greater the centrifugal force, according to the well-known formula, $F = \frac{m\,v^2}{R}$.

The problem of determining the amount of centrifugal force for any given velocity would be simple if the actual path of the moving load were known, but the curve followed by the load is not the curve derived from the static deflection, but that resulting from the static load plus the additional pressure due to centrifugal force.

This problem was fully worked out many years ago by Dr. H. Zimmerman, whose general conclusions and approximate formula are quoted in the writer's "Modern Framed Structures," Part II, page 524. A closely approximate formula for this centrifugal force for 60 miles per hour is:

$$F = P\,\frac{1}{0.595\,L - 3}.$$

For a span length of 25 ft., this centrifugal force amounts to 8.7% of the static load, and for a 50-ft. span, 3.7 per cent. This analysis, of course, assumes very precise conditions as to rigidity of supports, straight alignment of track, etc., and the results would be wholly upset were these conditions not very precisely fulfilled.

† The deflection of a short girder under live load may be taken at about $\frac{1}{1\,500} \times L$. Hence, energy used in acceleration $= \frac{P}{2} \times \frac{L}{1\,500}$. The increase in kinetic energy due to increase in velocity $= \frac{1}{2} M (v_2^2 - v_1^2)$, where M = mass of body $= \frac{P}{g}$, v_1 = velocity at the end of the beam, and v_2 = the velocity at the center. Hence, $\frac{PL}{3\,000} = \frac{1}{2}\,\frac{P}{g}\,(v_2^2 - v_1^2) = \frac{1}{2}\,\frac{P}{g}\,(v_2 + v_1)\,(v_2 - v_1)$. Since v_2 is only slightly greater than v_1, we can write $\frac{PL}{3\,000} = \frac{1}{2}\,\frac{P}{g}\,2\,v_1\,(v_2 - v_1)$. We then find that the increase in velocity, or $v_2 - v_1$, is approximately equal to $\frac{L\,g}{3\,000\,v_1}$, or, in foot-second units, $v_2 - v_1 = \frac{L}{100\,v_1}$ (roughly). For example, for L = 50 ft. and v_1 = 90 (60 miles per hour), $v_2 - v_1 = \frac{50}{9\,000} = 0.0055$ ft. per sec., or about 0.006% of the initial velocity.

It has always seemed to the writer, therefore, that the results of this analysis are of no practical value, and that, considering the relatively small percentages for even the shortest span, it may fairly be concluded that the effect of speed is of no practical consequence.

It appears to the writer, therefore, that while the author's final statement as to the unimportance of centrifugal force is correct, his analysis aiming to prove this is quite incorrect.

The second part of the paper omits the element of centrifugal force, but takes account of the effect of the mass of the beam by an approximate method. The results of this analysis show a vibratory effect, due to the elastic action of the beam. This analysis is interesting and useful in showing mathematically that the effect under ordinary working conditions in this case, also, is of no particular consequence.

There appears to be an error in the author's integration of Equation (4), shown in Equation (5). Equation (4) cannot be obtained from Equation (5) by differentiation, but if the term, $24\ C_1\ C_2\ C_3^3\ t$, is added to Equation (5), the work will check. Compared to the other term containing t, this additional term is very small, so that the effect of the error is insignificant.

F. W. GARDINER,* M. AM. SOC. C. E. (by letter).—Mr. Paaswell's discussion is a valuable addition to the record, and the history of the analysis of the problem given therein will be of great value to those interested in this subject. It will be interesting to compare the amplitude of the vibration at the center of the span given by St. Venant with that given in the paper. The expression given by St. Venant for the term representing vibration is:

$$-\frac{25}{96}\frac{l^3}{E\,I}\left[Q\left(1-\frac{2}{\beta}\right)+\frac{31}{84}\frac{P}{\beta}\right]\frac{r}{l}\sin\frac{l}{r}$$

where l is one-half the span length, Q the live load, P the dead load, $\frac{1}{\beta}=\frac{2\,Q\,l\,V^2}{3\,G\,E\,I}$,

and $\frac{r^2}{l^2}=\frac{31}{252}\times\frac{P}{Q}\times\frac{1}{\beta}$.

The expression given in the paper for the term representing vibration is:

$$-3\sqrt{C_2}\,C_1\,C_3\,L^2\sin\frac{t}{\sqrt{C_2}}.$$

The amplitude of the vibration can be derived from each of these expressions and put in the same form, namely,

$$A=-\text{(constant)}\,\frac{d}{N}\qquad\qquad(22)$$

where d is the center deflection due to a load at rest at the center, and N is the number of free vibrations of the beam during the passage of one axle across the span.

From the first of the foregoing expressions (St. Venant),

$$A=-\frac{25}{96}\frac{l^3}{E\,I}\left[Q\left(1-\frac{2}{\beta}\right)+\frac{31}{84}\frac{P}{\beta}\right]\frac{r}{l}\qquad\qquad(23)$$

* New York City.

Also,

$$d = \frac{Q\,l^3}{6\,E\,I} \qquad N = \frac{2\,l}{2\,\pi\,r} = \frac{l}{\pi\,r} \qquad \frac{1}{N} = \frac{\pi\,r}{l}$$

Equation (23) can be written as follows:

$$A = -\frac{1}{\pi} \times \frac{25}{16} \times \frac{Q\,l^3}{6\,E\,I}\left[1 - \frac{2}{\beta} + \frac{31}{84}\,\frac{P}{\beta} \times \frac{1}{Q}\right]\frac{\pi\,r}{l} \quad \ldots\ldots(24)$$

or,

$$A = -0.49\left[1 - \frac{2}{\beta} + \frac{31}{84}\,\frac{P}{\beta} \times \frac{1}{Q}\right]\frac{d}{N} \quad \ldots\ldots(25)$$

St. Venant states that the value of β must be not less than about 20, and it will be noted therefore that the value of the term in brackets is always somewhat greater than 0.9. For the cases worked out in the paper, it is about 0.92, and the amplitude according to St. Venant would be therefore

$$A = -0.45\,\frac{d}{N} \quad \ldots\ldots(26)$$

From the second of the foregoing expressions (that in the writer's paper for the term representing vibration),

$$A = -3\,\sqrt{C_2}\,C_1\,C_3\,L^2 \quad \ldots\ldots(27)$$

$$N = \frac{\frac{L}{C_3}}{2\,\pi\,\sqrt{C_2}} \quad \ldots\ldots(28)$$

$$d = \frac{P\,L^3}{48\,E\,I} = C_1\,L^3 \quad \ldots\ldots(29)$$

from Equation (28)

$$C_3\,\sqrt{C_2} = \frac{L}{2\,\pi\,N}$$

from Equation (29)

$$C_1 = \frac{d}{L^3}$$

substitute in Equation (27)

$$A = -3\,\frac{L}{2\,\pi\,N} \times \frac{d}{L^3} \times L^2 = -\frac{3}{2\,\pi}\,\frac{d}{N} = -0.48\,\frac{d}{N} \quad \ldots\ldots(30)$$

Inasmuch as these two results, Equations (26) and (30), were arrived at by totally different methods of analysis, they are in remarkably close agreement and afford a satisfactory check on the analysis given in the paper.

The exact form of the solution of the differential equation given by Mr. Paaswell is appreciated by the writer as an independent check on the analysis, and will answer the objection made by Mr. Mensch in his discussion.

The error made by Mr. Mensch in his analysis is the assumption of a constant acceleration of the mass of the beam during the passage of the load from the support to the center of the span, and his neglect of the effect of the energy stored in the mass velocity. The effect of increasing mass in a beam under given rolling load conditions is to increase the amplitude of the vibration and also the time length of the wave.

Professor Turneaure's discussion will make clear a point that seems to be obscure in the paper, namely, that the effect of centrifugal force was omitted in the paper, as stated at the bottom of page 437.

The writer intended the theoretical discussion in the first part of the paper to show that the total external work, Pd, done by a rolling load, P, falling through a deflection, d, is resolved into two components of equal value, $\frac{Pd}{2}$, one of which is effective in deflecting the beam and the other is absorbed in accelerating the mass of the load and is stored in kinetic energy.

The action of a rolling load accelerating and retarding over the curved path formed by the flexible beam is very much like the swing of a pendulum, and the kinetic energy accumulated on the down swing is available to propel the mass on the up swing, the action being entirely within the mass of the load and having no effect on the deflection of the beam except in so far as centrifugal force is developed.

This theoretical discussion has no relation to centrifugal force, the intention being to note that such a force did exist and then ignore it, because its effect had been determined by Professor Turneaure's well known annotation of Dr. Zimmerman's analysis.

The writer is indebted to Professor Turneaure for his discussion of the effect of centrifugal force, without which the subject would not be complete, and for his correction of the inaccurate use of the word "acceleration" in the statement he criticizes. It was the intention only to note that a centrifugal force was developed by the load being constrained to move in a curved path.

The writer desires to acknowledge the added value given the paper by Professor Turneaure's discussion.

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Paper No. 1472

CONTROL OF FLOOD AND TIDAL FLOW IN THE SACRAMENTO AND SAN JOAQUIN RIVERS, CALIFORNIA

By C. S. Jarvis,* M. Am. Soc. C. E.

With Discussion by Messrs. C. E. Grunsky, H. H. Wadsworth, Fred. H. Tibbetts, and C. S. Jarvis.

Synopsis.

This paper briefly reviews the progress made thus far toward flood control on the Sacramento and San Joaquin Rivers, California, and describes the new conditions obtaining along the lower courses of these rivers, which seem to demand the exclusion of the tidal effects from these channels.

The intricate relationship between navigation, irrigation, land reclamation, drainage, water supply, and timber preservation in this district and the varying salinity of the Upper Bay and lower river channels are discussed, together with proposed methods of controlling the existing menace.

Essentially, the paper consists of a presentation of the outstanding features of one of the most important engineering and economic problems in California, with a basic solution proposed and supported by examples and precedents.

About 1 000 000 acres of the most productive valley of California, the Great Valley of the State, probably as fertile as any on the globe, are alternately laid waste by inundation and then parched by drought because the floods and tidal currents are uncontrolled; and now that the oldest established and choicest farm tracts along the river front have been provided with adequate levees for flood protection and pumping plants for irrigation and drainage, and the yields of fruits and vegetables from the deep alluvial soil have established new records for production, the salt water from San Francisco Bay has invaded the river courses and rendered the pumping plants for irrigation entirely useless in midsummer. With no fresh water available for surface supply, the continued presence of the salt water in these channels will inevitably, by subsurface irrigation, impregnate parts of these river-front farms with ruinous proportions of alkaline elements.

The incursions of saline water have been prevented heretofore by the strength of the current from the minimum flow of the Sacramento and San Joaquin Rivers;

* Capt., Corps of Engrs., U. S. A., Camp Lewis, Wash.

probably 3 000 to 4 000 sec-ft. of steady flow is sufficient for this purpose. In other words, the regulating dam that has served to exclude the salt water from this district has consisted of at least 3 000 sec-ft. of fresh water, which has also maintained the navigation interests along the two rivers to the extent of a 7 or 8-ft. draft at low tide.*

Elements of the Problem.

The great interior valley of California, lying between the parallel Sierras and the Coast Range, is watered and drained from the south by the San Joaquin River, and from the north by the Sacramento River. Meeting at a point about midway between Sacramento and San Francisco, these two rivers flow westward through drowned valleys known as Suisun Bay, Carquinez Straits, San Pablo Bay, San Pablo Straits, and the north end of San Francisco Bay, thence through the Golden Gate.

At high flood flow the Sacramento River has discharged in excess of 600 000 sec-ft. from a water-shed of 27 000 sq. miles, thus ranking after the Mississippi, the Columbia, and the Ohio Rivers of the United States; and the San Joaquin River, with more than double the drainage area and about one-half the precipitation, has been observed to discharge 325 000 sec-ft.†

The natural channels of these rivers below Sacramento and Stockton accommodate perhaps one-fourth of the maximum flood flow and below the confluence the width of definite channel is approximately 3 000 ft., with an average depth of 30 ft. at mean tide. Taking into account the very flat gradient of the river, and the fact that the mean tidal range at Sacramento is 0.8 ft., at Stockton 2.0 ft., and at the junction of the two rivers 4.0 ft., it is evident that the maximum velocity attainable during the highest floods would account for not more than one-half of the combined flows in the main channel below the junction. The remaining 50% either flows over the adjacent lowlands, extending 2 miles northward and ½ mile southward, or impounds temporarily on the rich alluvial areas.

With the construction of levees to protect the farms in the island district, near the junction of the two rivers, has come the necessity for the floods to extend farther afield in the unreclaimed areas.

Since 1862, when the overflow formed a navigable lake from Sacramento to Stockton and westward to the sea during high flood, the enormous problem of regulating the floods and channels of these rivers has been officially recognized, and surveys and recommendations have been made by the State Department of Engineering. The four-sided conflict between the navigation, irrigation, reclamation, and power interests is now in process of solution under the able direction of a Reclamation Board and the California Débris Commission, comprised of representatives of the State and of the War Departments, respectively, and working in conjunction with the State Department of Engineering.

Comprehensive plans devised and largely executed by these agencies include the enlargement and stabilization of the river channels, and the establishment of auxiliary channels, or by-pass areas, over which the crests of floods will flow unobstructed. After March of each year, when the danger of recurring floods has

* A flow of 7 000 sec-ft. for navigation was regarded as desirable in the report of the California Conservation Commission, 1912.

† Fifth Biennial Report, Dept. of Eng., State of California, 1916, p. 37.

passed, the land owners may safely plant crops in the by-pass areas; but no permanent obstruction is allowed.

The Sacramento River, like the Mississippi, has built banks higher than the adjacent country; the resulting valleys parallel with the river channel are obviously the passages chosen for the flood crests to traverse. Incidentally, these by-pass channels intercept the lateral drainage and silt from the foot-hills, and conduct it to the lower courses of the river near the junction. The San Joaquin River occupies the lowest part of its valley, but the high flood crests are disposed of between the protecting levees of reclaimed areas along lines prescribed by the Reclamation Board.

The rapid development of irrigation projects in the San Joaquin and Sacramento Valleys recently, coincident with a series of dry years, has reduced the flow of water at the mouths of these two rivers to approximately 500 and 1 500 sec-ft., respectively, in midsummer during the last two years. The natural result of this decreased flow, together with channel improvements in the interest of navigation, is that the tidal currents extend farther and farther up the river channels, introducing salt water into sections of the river never before affected.

During August, 1920, the range of tide level at Sacramento increased to nearly 3 ft., with only 1 300 sec-ft. passing the city, with the low-water level at 0.5 ft. on the gauge, the zero of which is at mean sea level. It is thus apparent that under such conditions in the 61 miles of river channel from Sacramento to Collinsville, near the mouth, the slope of the water surface is from 2.5 to 3.5 ft., or about 0.5 in. per mile. Under maximum flood conditions, the average slope between these points may increase to 6 in. per mile.

Prior to 1918, the Sacramento River maintained such a volume at minimum flow as to protect the lower reaches of both rivers, the San Joaquin low flow having been completely utilized by diversion for several years. Now that the diverted waters from both rivers exceed the flow during midsummer, however, the discharge into Suisun Bay at such times actually deprives the recent appropriators of water of their means of subsistence. Recognizing that encroachments of salinity would ruin the farms possessing the oldest vested water rights, and deprive thriving communities of their municipal water supply, it is inevitable that litigation should be resorted to for determining the relative rights; it has been impending for the past two years and is now in process.

The section of the Sacramento River now affected with salt water was seriously considered as a source for San Francisco water supply as late as 1911* and for Vallejo and Mare Island until the saline invasion of 1918. The trouble now developing on account of encroaching salinity seems to afford convincing proof of the wisdom displayed in seeking mountain sources for the domestic water supply for San Francisco and other Bay cities.

The salt-water menace will doubtless become more accentuated in succeeding years, even with the same conditions of flow; and if the entire yield of the two rivers should be diverted, the salinity would extend far above Stockton and Sacramento.

* "San Francisco Water Supply", Report by John R. Freeman, M. Am. Soc. C. E., 1912, p. 317, dealing with the report on "Filtered Water Supply for San Francisco", by Allen Hazen, M. Am. Soc. C. E., Consulting Engineer. Rio Vista was the chosen location for the intake.

The quantity of saline elements in ocean water is approximately 3%; and the allowable percentage for irrigation in these localities may be taken at from $\frac{1}{30}$ to $\frac{1}{100}$ of this amount; so that an admixture of 1 unit volume of sea water with 30 such volumes of fresh water would render the resulting mixture unfit for irrigation purposes. For domestic supply the saline content should be much lower, generally below 100 parts per million of total dissolved salts in humid districts, or 200 parts in arid sections. With a tidal prism of from 50 000 to 150 000 acre-ft. in Suisun Bay, a mean tidal variation of 5 ft. between lower low water and high water, an extreme range of 9 ft., and river channels being constantly improved while the discharge is decreasing, the results produced thus far in extending the domain of saline water are but natural, and can be avoided only by positive control of the fresh water or of the sea water.

Early Solution of Problem Imperative.

To the whole State of California the importance of an early solution of the difficulty is imperative. Obviously, the possible corrective measures that may be applied would be either to maintain a larger minimum flow of fresh water in the rivers, or else to exclude the tidal currents from these channels.

Assuming that it is required to augment the natural flow of the Sacramento and San Joaquin Rivers by 3 000 sec-ft. for 100 days each year, this would necessitate the retention of 600 000 acre-ft. of water in storage reservoirs, or the deprivation of the irrigation interests of this quantity. Further, assuming that the water delivered from reservoirs for this purpose or conducted away from canal intakes represents the moderate value of $5 per acre-ft., or 1½ cents per 1 000 gal., it is evident that $3 000 000 per year would be represented in the cost of combatting the encroaching salinity and maintaining the required depth of water in channels for navigation. It is probable that one or perhaps two yearly payments as previously named would finance the construction of a dam equipped with a movable crest and multiple locks between Suisun Bay and the junction of the two rivers. By widening the channel to compensate for the piers and sills of the dam and locks, and by having the lock-gates open and the crests removed during the flood season, similar to the Ohio River improvement, no appreciable retardation of the flood current would result from this structure. Furthermore, it will be recalled that about one-half of the highest flood crests flows over the adjacent lowlands, beyond the influence of the proposed structure. During the low-water season it would seem advantageous to keep the water surface above the locks at or above high-tide level.

The advantages that may be claimed for such a plan are:

1.—The increased depth of navigable channel for a distance as far as backwater would reach; this would be of benefit for some distance beyond Sacramento and Stockton.*

2.—A positive barrier would be formed against the incursions of salt water, even the leakage about the gates always being from the fresh-water channel.

* The Fifth Biennial Report, California State Department of Engineering, p. 9, states "It is engineeringly feasible to construct tidal canals capable of conveying ocean-going ships to Sacramento and Stockton, at a comparatively low cost."

3.—The agricultural development of the rich interior valleys would be encouraged and assured, to the fullest extent, free from the uncertainty now brought about by the conflicting interests.

Alternative Sites for Locks and Dam

Obviously, the construction of similar controlling structures at the Straits of Carquinez or Point San Pablo (Figs. 1 and 2) would accomplish the main purpose of excluding salt water from the river; but it is evident that the highest surface elevation that could reasonably be advocated for the Bays would be mean high water, while the river section would be benefited by storing at a considerably higher level during the low-water season.

The reclaiming and draining of low tidal lands would be best served by maintaining the surface as near low-tide level as practicable. On the other hand, the irrigation of reclaimed lands is facilitated by keeping the water surface relatively high, to permit siphoning instead of pumping for irrigation; and as this high stage always recurs during flood seasons, say, from December to March, it would appear best to maintain the water surface of the Bays at about high tide in the early part of the irrigation season, and to release the surplus during the period of danger from floods.

One disadvantage of proposing alternative dam sites for the purpose of checking the tidal currents at Carquinez Straits or Point San Pablo, or near the Golden Gate, is the prevailing idea that a dam at one of the lower sites would render the structure at the mouth of the combined rivers unnecessary. The need of a deeper channel for navigation, however, would probably justify the adoption of a storage level within the river channels from 5 to 10 ft. above highest tide, as all the river banks, levees, and shipping wharves are built to accommodate such fluctuations, or greater, the range at Stockton and Sacramento being about 15 and 30 ft., respectively.

It appears reasonable that a moderate storage depth in the main river channels, simulating the conditions that prevailed along these river banks until recent years, should be of great advantage to navigation and irrigation interests alike. Not the least of these advantages would be the storage of about 20 000 acre-ft. of fresh water for each foot of added depth. The maintenance of a constant level during the growing season, from March until November, would assist in protecting the banks against erosion by establishing a dense growth of willows, which thus far has proved as effective as any revetment yet provided, or one of the most effective. The increased depth of water in the river channels would add somewhat to the expense of draining the land and would probably justify the construction of drainage canals parallel to the river, discharging directly into Suisun Bay.

Conclusions.

From a comparison of the cross-sections of the various sites that have been considered for regulating dams and locks (Figs. 3 and 4), it will be plainly seen that all projects except the one at the head of Suisun Bay are of such gigantic proportions that there is no agency except the Federal Government that should undertake any one of them, and it is questionable whether the conflicting interests can be brought to agreement on a definite plan.

SONOMA
NAPA
SOLANO
MARIN
CONTRA COSTA
ALAMEDA
SANTA CLARA
Petaluma
Petaluma Creek
Napa
Napa River
Suisun City
Suisun Cr.
Montezuma
GRIZZLY ISLAND
Vallejo
MARE ISLAND
Army Base
Benicia
SUISUN BAY
Carquinez Strait
SAN PABLO BAY
Port Costa
Martinez
Suisun Pt.
Sacramento R.
SHERMAN ISLAND
San Joaquin R.
San Pedro Pt.
San Rafael
San Pablo Pt.
Pt. Richmond
Berkeley
MARIN PEN.
Sausalito
Tennessee Cove
Pt. Bonita
Pt. Lobos
San Francisco
Oakland
Alameda
SAN FRANCISCO BAY
South San Francisco
Redwood City
Palo Alto
Santa Clara
San Jose
PACIFIC OCEAN
SAN MATEO
MAP OF
SAN FRANCISCO
AND VICINITY
Channel Sections Examined and Included on Profiles.
SCALE OF MILES

FIG. 1.

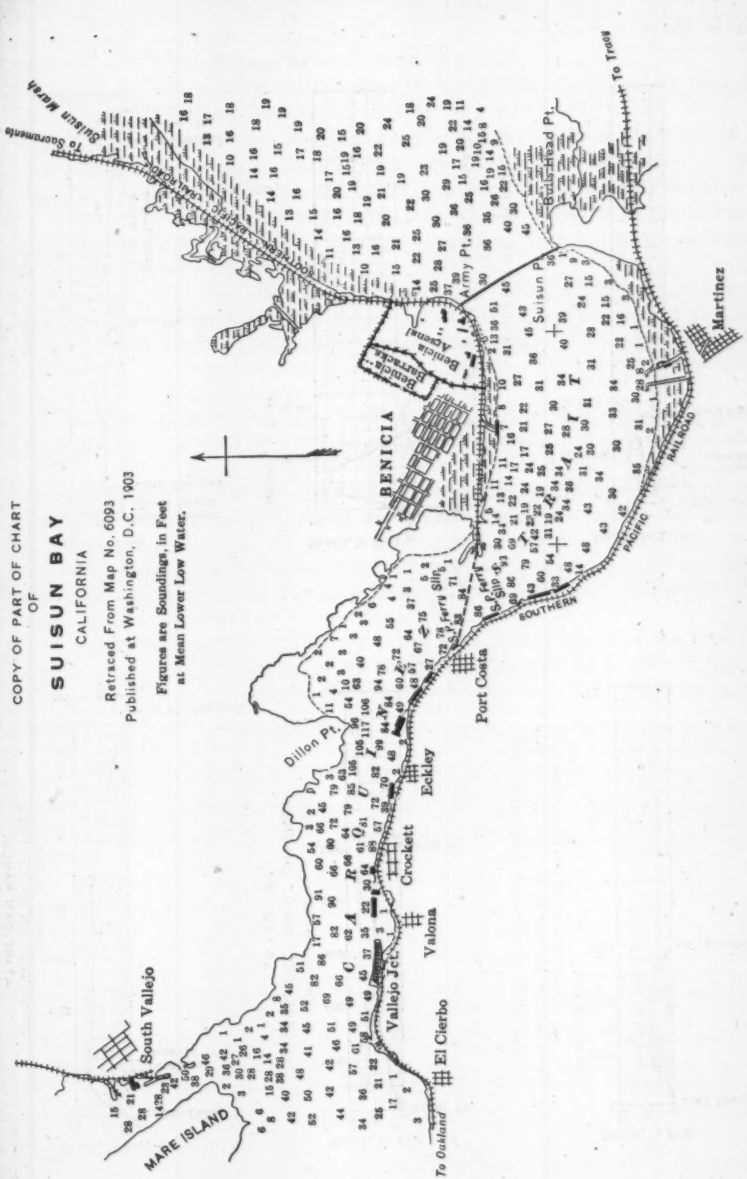
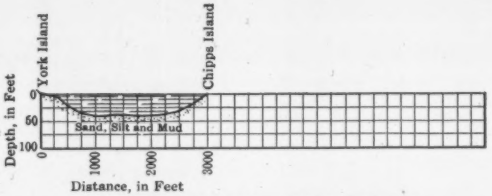
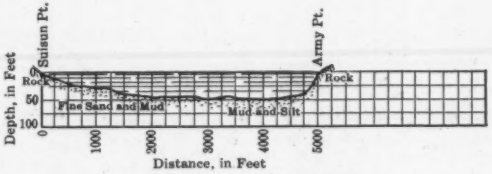

COPY OF PART OF CHART
OF
SUISUN BAY
CALIFORNIA
Retraced From Map No. 6093
Published at Washington, D.C. 1903
Figures are Soundings, in Feet
at Mean Lower Low Water.
BENICIA
Benicia Barracks
Benicia Arsenal
Army Pt.
Suisun Pt.
Bull Head Pt.
Martinez
To Sacramento
Suisun Marsh
To Tracy
SOUTHERN PACIFIC RAILROAD
Port Costa
Eckley
Crockett
Valona
Vallejo Jct.
El Cierbo
South Vallejo
MARE ISLAND
To Oakland
Dillon Pt.
CARQUINEZ
TRAIT

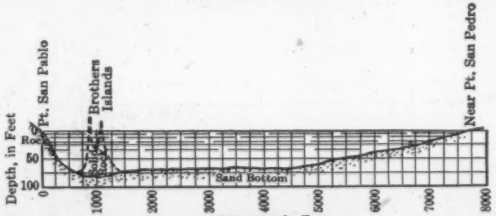
FIG. 2.



CROSS SECTION OF RIVER CHANNEL
TWO MILES BELOW THE JUNCTION OF THE
SACRAMENTO AND SAN JOAQUIN RIVERS.

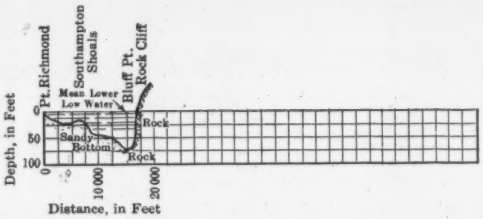


CROSS SECTION OF CHANNEL
AT ARMY PT.

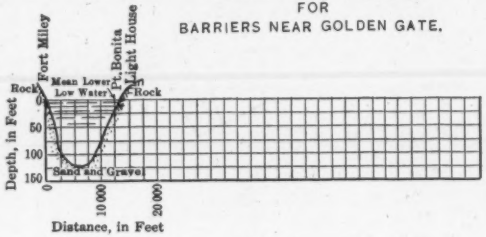


CROSS SECTION OF CHANNEL
AT PT. SAN PABLO.

FIG. 3.



COMPARATIVE PROFILES
FOR
BARRIERS NEAR GOLDEN GATE.



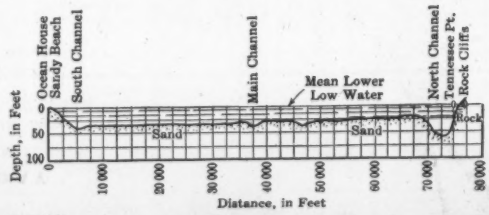


FIG. 4.

To finance this undertaking, there are so many interests that would be directly benefited that it will require wise adjustments of cost.

The rice growers could better afford to finance the whole project than to lose their anticipated profits from the $20 000 000 crop now maturing.

The municipalities now deprived of water supply, or the farmers in the island district of the delta region, could better afford to build such a structure than to continue two more years under the growing menace of salt-water intrusions.

New reclamation projects that have spent millions thus far in preparing tracts for irrigation could better undertake this work alone, than to lose the well deserved profits from their enterprises.

If the Cities of Sacramento and Stockton are seriously contemplating a deeper navigation channel, this would be a step in that direction, and it is almost a certainty that it would bring the most direct results for the least expenditure; the tortuous channel, however, would not likely be changed by this project, nor would a modified alignment be hindered.

The members of the Engineering Profession, in line with their special training in these matters, should be able to suggest a basis on which the various beneficiaries could unite in building a dam, equipped with locks and movable crests, below the junction of the Sacramento and San Joaquin Rivers. Perhaps the legal conflict now under way may require more time and financial support than would be required to build a concrete structure designed to pass the maximum flood without injury to any one, conserve the fresh water, exclude the tidal currents from the river channels, and develop the same navigable depth of channels to Sacramento and Stockton as now governs in Suisun Bay.

The principal purpose of this paper will have been achieved if the necessity for a regulating dam with locks near the mouth of the combined rivers has been established. Some data are appended which may be of value, if only suggestive, for use in the discussion.

APPENDIX

1.—Areas and Storage Capacities.

From planimeter measurements of the U. S. Coast and Geodetic Survey charts, the writer has obtained the data of Table 1 for various tracts in the districts considered.

TABLE 1.

Description of tract.	Area at mean high water, in acres.	Storage capacity at mean high water, in acre-feet.	Area with depth of 10 ft. or more at lower low water, in acres.
Suisun Bay, above Army Point	31 300	460 000	9 500
San Pablo Bay and Carquinez Straits	86 200	1 273 000	27 400
Point San Pablo to Point Richmond	16 000	480 000	11 200
Point Richmond to Golden Gate and southward to Point Avisadero	59 000	1 980 000	36 200
San Francisco Bay southward from Point Avisadero	109 600	1 924 000	33 200
Outside the Golden Gate to crescent-shaped bar	24 500	1 962 000	24 000
Totals	326 600	8 029 000	141 500

2.—Average Yearly Yields.

The average yearly yield of the Sacramento River and its tributaries may be taken at 27 000 000 acre-ft., and of the San Joaquin River at 14 000 000 acre-ft. According to the report of the State Reclamation Commission, 1912, there are available storage reservoir sites that when fully developed will yield about 3 400 000 acre-ft. total on these river systems; and when the development has been completed as contemplated, there will be a total irrigated area in the Sacramento Valley of 2 500 000 acres, and in the San Joaquin Valley of 4 000 000 acres; but during the average year nearly 10 000 000 acre-ft. of water will flow from these rivers into Suisun Bay, in spite of all storage and diversions.

3.—Areas Could Be Served by Pumping.

There are areas adjacent to San Francisco Bay and its tributaries aggregating more than 400 000 acres that could be profitably served with water by pumping, if the salt water were displaced by fresh water, at the present charges for electric current (about 1.5 cents per kw-hour for large installations), and it can be safely ventured that the exclusion of salt water from various arms of the Bay, and the retention of the winter run-off in its place, will proceed for this purpose under private financing in the near future in line with the present trend of development.

4.—Reclamation of Tidal Flats.

The reclamation of tidal flats at or near sea level in less favored locations may be comparable to reclaiming of vast areas around San Francisco Bay, San Pablo Bay, and Suisun Bay that are slightly below lowest tide level. More than half the total areas of these three Bays lies under less than 10 ft. of water at low tide. The most favored areas can be reclaimed by building 1 lin. ft. of levee for each acre of land, the surface of ground varying from 10 ft. below low-water level to 5 ft. above.

5.—Ravages of the Teredo.

The ravages of the teredo in timber piling and structures around San Francisco Bay, limiting the life of such works to less than a year if not thoroughly creosoted, have extended since 1917 to Carquinez Straits and Mare Island, endangering more than $5 000 000 worth of structures that were heretofore immune from this danger. It is well established that the teredo has merely kept pace with the incursions of salt water, and that a permanent freshening of the water so that less than 0.5% of sodium chloride is held would destroy these marine borers. In only isolated cases and very limited numbers and periods will the teredo maintain life and activity in fresh water.

6.—Dam to Reduce Tidal Prism.

The tidal prism within the Golden Gate, to cover more than 300 000 acres to the varying depths, ranges from about 1 000 000 to 2 500 000 acre-ft. Passing through a rock-bound channel, 5 000 ft. wide, with an average depth of 250 ft., in the course of 5 to 7 hours for a given phase of the tide, the currents may attain a surface velocity during spring tides of 8 ft. per sec. for short periods. At this rate the discharge through the Golden Gate is probably from three to five times the maximum flood discharge of all the tributary fresh-water channels. A dam equipped with tide-gates and locks from Point Richmond to Bluff Point on the Tiburon would reduce the tidal prism by 44%, and moderate the currents in the Golden Gate and also on the north shore of Angel Island.

7.—Deeper Channel Outside the Golden Gate.

The necessity for a deeper channel across the bar outside the Golden Gate is imperative in the near future, if the largest battleships and merchant vessels are to be accommodated. At low tide the depth on this bar is about 32 ft.; while 40 or 45 ft. is desired. It is proposed to establish and maintain this channel by dredging. Now, the largest vessels either must incur serious risks by taking the north or south channel, or wait for high tide and take the middle channel. The north channel has a governing depth of 54 ft., the main channel has 32 ft. minimum, with a distance of 4 000 ft. between 35-ft. contours, and the south channel has 36 ft. Due to the strong currents and bold headlands, together with the fogs that hover along the coast periodically, it is advisable to improve and use the main channel which gives a direct course through the Golden Gate.

8.—How to Preserve Navigable Channels.

Surveys and charts by Commander Cadwalader Ringgold, U. S. N., made in 1849 and 1850, together with more recent surveys by the U. S. Coast and Geodetic Survey, indicate that the bar and Four Fathom Bank outside the Golden Gate have remained for 70 years practically unchanged, although the depths and shore lines in San Pablo and Suisun Bays have been considerably altered by the deposits of sediment and débris from hydraulic mining operations and from natural erosion. Not the least of the difficulties to be reckoned with in the future control of floods and maintenance of channels is the disposal of the millions of cubic yards of material temporarily arrested along the courses of tributary streams. Under the able direction of the California Débris Commission, and related agencies, the heaviest

sediment and rock waste has thus far been largely excluded from the navigable channels; but the menace continues and will require wise control in future years and operations under a comprehensive plan such as the present one.

9.—Some Examples and Precedents.

For removing an extensive sand bar southeast of Heligoland on the Weser River, Herr Franzius constructed 5.3 miles of training walls at well chosen locations, and in two years the floods and tidal currents displaced nearly 12 000 000 cu. yd.*

The training jetties at Tampico Harbor, Mexico, extending for more than a mile from shore into a 24-ft. depth of water, were built by late Elmer L. Corthell, Past-President, Am. Soc. C. E., in accordance with the practice at the mouth of the Mississippi River; and the dikes described by W. M. Black,† M. Am. Soc. C. E., at the New Inlet, North Carolina, are precedents for such training works as may be found advisable on the sand-bars or the mud or sand foundations in the Bay region of California.

Regarding depths, the breakwaters constructed at Dover and Holyhead, England, extend into 45 and 50-ft. depths below lowest tide level; while 55 ft. at Marseilles; 70 ft. at Algiers; 89 ft. at Sandy Bay Harbor of Refuge, Massachusetts; and 101 ft., at Naples, seem to be the limiting depths in which such controlling works have been constructed.‡

The foundations for the mole or sea-wall at Zeebrugge, Belgium, were constructed in separate segments about 82 ft. long, 24.5 to 29.5 ft. wide, and 23 to 30 ft. deep, in the form of concrete caissons or box-shaped barges which were floated to position, anchored, and sunk, and then filled with ballast of concrete. Upon this structure the 55-ton concrete blocks were placed above low-water level.

Considering such precedents, any one of several sites that have been examined and discussed, between the bar outside the Golden Gate and the junction of the Sacramento and San Joaquin Rivers, would present no unusual difficulty. However, the growing importance of San Francisco Bay as a harbor, a commercial center, and a naval base, precludes a serious consideration of any such structure south of Point Richmond. Inland from this point it would warrant investigation concerning the relative advantages of transformation into a fresh-water bay, undisturbed by tidal fluctuations, with an increased depth of 3 to 6 ft., and with timber structures relieved of the destructive marine borers, as compared with the disadvantages of passing every vessel through locks, and the requirement that floods be discharged without producing back-water against the levees of reclaimed lands.

10.—Other Similar Engineering Achievements.

On account of the swift currents and the dangers to navigation in traversing the reversed curves of the River Scheldt in the vicinity of Antwerp, Belgium, a new channel and a shipping basin provided with locks for maintaining the water at high-tide level, were constructed by Mr. Lindon W. Bates,‖ of New York City.

* *Transactions*, Am. Soc. C. E., Vol. XXIX (1893), p. 179; "Harbor Engineering," by Brysson Cunningham, p. 312.

† *Transactions*, Am. Soc. C. E., Vol. XXIX (1893), p. 242.

‡ "Harbor Engineering", by Brysson Cunningham.

‖ *Engineering Record*, May 20th, 1905.

There seemed to be no especial objection to the use of the same method on a section of the Thames River, where the main advantage, if not the only one, was the securing of an increased depth of channel.

Of the many notable achievements of the late Frederic P. Stearns, Past-President, Am. Soc. C. E., it is doubtful whether any has brought more wholesome pleasure into the lives of a community than the plans for the improvement of the 11-mile section of the Charles River Basin, Boston, Mass. Due to sewage pollution and the periodic exposure of the tidal flats at low tide, Mr. Stearns planned to create a fresh-water basin to be maintained at a constant level, 2.5 ft. below high tide, the dam to be equipped with tide-gates and openings to discharge the flow from the uplands, and with lock capacity for the largest vessels that would ply on this lake. In the apt words of Leonard Metcalf, M. Am. Soc. C. E., in the memoir of Mr. Stearns*:

"* * * the foul estuary of the Charles River, with its tidal flats, was converted into a beautiful fresh-water basin, completely changing the character of the district and becoming one of the great recreation waterways and parks of the United States."

11.—Proposed Dam.

The New York State Barge Canal, with length and elevations comparable with those of the river channels connecting Red Bluff and Bakersfield, Cal., the Ohio River improvement, including fifty-three dams and locks to regulate the minimum flow and to pass the maximum flood, and the Assuan Dam on the Nile, designed to pass a flood of 490 000 sec-ft. and store 800 000 acre-ft. for irrigation, all accomplished their respective purposes creditably, each at a cost far in excess of what it would require to accomplish all three objects in the Great Valley of California.

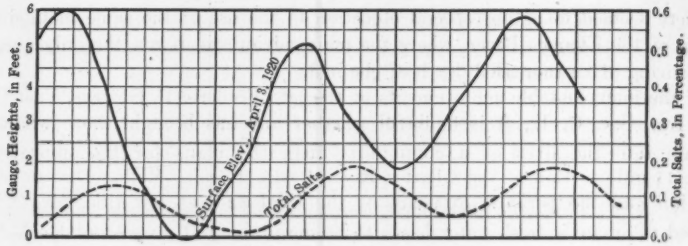
A dam with tide-gates, adequate sluice-ways for floods, and locks either at Point Richmond or Point San Pablo would store between tide levels about the same volume of fresh water as the Assuan Dam controls; and while retaining any part of such storage, would effect a deepening of the navigation channel, as well as the benefits of a fresh-water bay, and would furnish an acceptable highway with short-span bridges at the locks for crossing an expanse of water as yet unbridged for 85 miles.

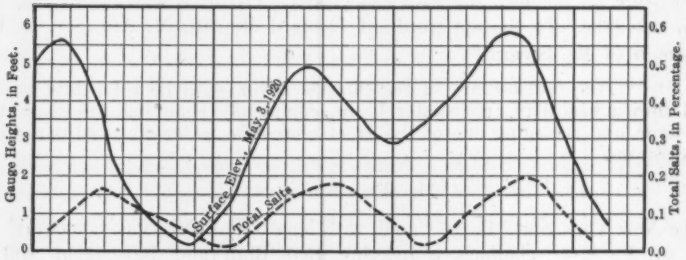
12.—Saving by Use of Surplus Waters.

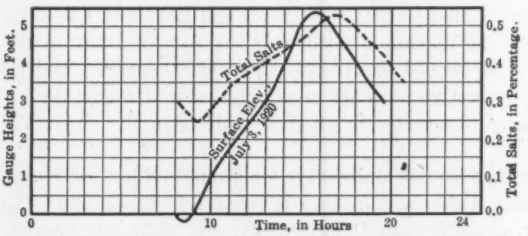
The people of Benicia, Cal., are to-day paying from $1.50 to $1.30 per 1 000 gal. for water, delivered by barge from the Sacramento River above salt-water intrusions; and the municipal supply is necessarily limited by the barge capacity.

While on duty at Benicia Arsenal as Assistant Constructing and Utilities Officer, the writer made careful studies of the saline content in the Bay water, and having learned the relation of the salinity to the phases of the tide, as portrayed on the curves in Fig. 5, full advantage was taken of the fresh water whenever it prevailed, to the limit of the current needs and the existing storage reservoirs. Nearly every day from March 15th until June 22d, 1920, the total saline content reduced to less than 0.1% for 1 hour or more at Army Point; and for daily require-

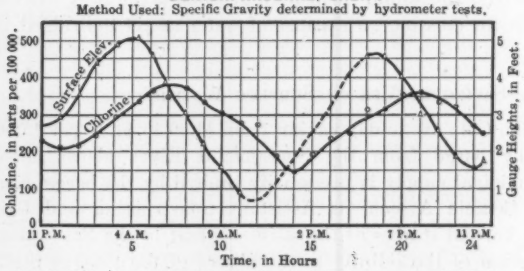
* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2134. This project is also referred to in *Engineering News*, May 31st, 1894, and August 16th, 1894, p. 131.







RELATION OF TOTAL SALINE CONTENT TO TIDAL CONDITION.
OBSERVATIONS TAKEN AT ARMY POINT
BENICIA ARSENAL, CALIF.
Method Used: Specific Gravity determined by hydrometer tests.



RELATION OF CHLORINE CONTENT WITH TIDAL CONDITION
AT
ANTIOCH (H St. WHARF)
Aug. 2, 1920.

Data Furnished Through Courtesy
of California State Water Commission.

FIG. 5.

ments and filling all existing storage reservoirs, a total of 6 400 000 gal. of water was pumped at a power cost of less than $200. At the prevailing price at Benicia, this water would now cost $9 600.

Fresh water has been pumped from Carquinez Straits at the Crockett Wharf for the Sugar Refinery.*

The surplus waters of the Sacramento and San Joaquin Rivers which cannot be retained in mountain reservoirs, should be made to displace the salt water of Suisun and San Pablo Bays, and maintain a fresh-water supply for this region throughout the year.

* Report on the San Francisco Water Supply, by John R. Freeman, M. Am. Soc. C. E., p. 315.

DISCUSSION

C. E. GRUNSKY,* M. AM. SOC. C. E. (by letter).—In this paper the author makes a suggestion which is worthy of serious consideration. The writer's remarks will be brief and will be addressed mainly to the broad question of whether, if economically feasible, the project of a dam which would convert the northern portion of the San Francisco Bay system into a fresh-water lake, would be desirable.

Under natural conditions, the valley sections of the Sacramento and San Joaquin Rivers were flanked by broad areas of land which were submersible at flood stages. Along the Sacramento River these areas took the form of flood basins or depressed areas of large extent. The two great parallel depressions on either side of the river are subdivided by the sedimentary ridges of tributary streams into a number of separate areas or basins. Thus, on the west side of the Sacramento River there is Colusa Basin to the northward of Knights Landing Ridge (built by Cache Creek), and Yolo Basin to the southward of this ridge. On the east side of the river there is Butte Basin, above the Marysville Buttes, Sutter Basin, south of the Marysville Buttes, lying between the Sacramento and Feather Rivers, and American Basin extending from the Feather River to the American River. To the south of the American River is another natural, less pronounced basin sometimes called the Sacramento Basin. In their natural condition all these basins served as retarding basins, decreasing the peak of floods so that, except under extreme conditions, the maximum rate of discharge into the head of Suisun Bay was kept at probably from one-third to one-half of what it may be hereafter when the basin lands have been reclaimed to the full extent proposed.

The flood-control project for the Sacramento Valley now nearing completion is based on the following basic principles:

1.—To confine as much water to the river channel as can be carried by the river within levees of reasonable height.

2.—To allow water in excess of the high-water capacity of the river, to escape at selected points under control.

3.—To keep the escaping flood waters under control between high embankments on their course to a re-entry into the lower reaches of the river.

4.—To give the lower reaches of the river adequate capacity to carry the resulting increased flood flow.

Whether or not the idea of eliminating the flood basins, and thereby boosting the peak of the flood, has been carried to an extreme, is an open question which need not be discussed at this time. The elimination, however, of the major portion of the up-river flood basins, and the reclamation of about 400 000 acres of delta lands, of which more than one-half had originally been subject to submergence at the high tides of low-water stages, has accelerated the delivery into the Bay of the run-off resulting from winter rains and snows and has thereby contributed to the greater salinity of the Bay waters. It is to be remembered in this connection that in California there is so little rainfall from May 1st to November 1st, that

* San Francisco, Cal.

its effect on run-off is ordinarily negligible, and that, in consequence, the rivers have a pronounced and long sustained annual low-water period.

The use of water for irrigation has gradually become a factor which affects in some degree the low-water discharge of the two rivers into the Bay. The delta region of these rivers to the extent of about 300 000 acres has been brought under irrigation within comparatively recent years. Still more pronounced has been the increase of the up-river consumption of water for irrigation, due largely in the last few years to stimulation resulting from the high prices paid for food products during and immediately after the World War. The irrigation draft on the rivers and their tributaries has become such that in climatic years of light rainfall and less than normal run-off, such as 1919-20 and possibly, also, 1918-19, at low-water stages, the available fresh water has been barely able to meet the demand. At times the flow at flood tides up stream into the rivers from the Bay has exceeded by some quantity the return flow at ebb tides. During the prevalence of such a condition there is no material accession of fresh water to the Bay. The only river water getting into the Bay then is that which is displaced by Bay water in the process of mixing which results from the tidal movement in the lower reaches of the rivers.

This condition has led to the contention by the authorities of the Town of Antioch, situated within a few miles of the mouth of the San Joaquin River, that the up-river irrigators have no right to deplete the flow of the river below a point at which its lower reaches become more saline than they would under natural conditions. The water at the head of Suisun Bay under this contention should be kept fresh practically throughout the year by materially restricting the use of water at up-river points. It is not necessary to discuss the merits of this claim, under which a large volume of water would be withheld from use at full efficiency to serve at low efficiency in keeping the water of the lower rivers fresh. This situation, however, is referred to in order to make clear the advantage that would result from checking the salt water at some point, preferably as far down stream as the lower end of San Pablo Bay. The merit of the author's paper lies in pointing out this fact.

A dam at the lower end of San Pablo Bay would convert a salt-water area of about 100 000 acres into a fresh-water area. The flanking tide-marsh lands would be surrounded by fresh, instead of by salt and brackish, waters. Fresh water now available in insufficient quantities for industrial purposes would be made abundant on both shores of San Pablo and Suisun Bays and along the Straits of Carquinez, greatly benefiting such towns and cities as Benecia, Vallejo, Fairfield, Suisun, Martinez, Port Costa, Richmond, Berkeley, and even Oakland. The two Bays would become a storage reservoir in which in the spring of the year at the close of the snow-melting period the water surface would be held at about 6 ft. above ordinary low tide. Evaporation, navigation requirements, and losses would lower the water surface possibly from 2 to 3 ft. in the course of the summer and fall months, leaving from 3 to 4 ft. of water in the reservoir for use by the irrigators of the delta lands already reclaimed and under cultivation and for use on the tide lands the reclamation of which is not yet so far advanced.

The value of such storage, accessible to several hundred thousand acres of salt marsh and to an area of delta lands which has an extent of from 400 000 to 500 000 acres, can readily be appreciated. These lands lie along a network of channels

which are practically at Bay level during the low-water period of the year and which at once would become the distributaries of the reservoired water. The land surface of about one-half the delta area is so low that the water will flow by gravity through culverts or through siphons from the channels to the irrigating ditch. The remainder of the area is readily commanded by pumps. There are already installed for irrigation, in this area, about 650 pipe siphons, with diameters of 6 to 14 in. and about 370 pumps with a wide range of capacities. The deficiency of the river flow at the low-water period, if a Bay storage project were carried out, would thus be made good out of the storage. Furthermore, the low-water navigable depth in the lower rivers and in Suisun and San Pablo Bays would be increased; and the controlling or storage structure could be planned so as to make highway and rail connection possible between the south and the north shore of the north Bay region.

Recent investigations by the San Francisco Bay Marine Piling Committee of the American Wood Preservers' Association, relating to the damage to timber structures which has been done within the last few years by the teredo in the northern portion of the San Francisco Bay system, have led the Committee to conclude that this damage, attributable to marine borers and stated by the Committee to be evident only within the last two years, estimated in terms of replacement and repair, has been in excess of $15 000 000. If the salt and brackish water of the Bays was replaced with fresh water, untreated timber could be used in these waters without fear of attack by marine borers. An ample fresh-water area would be provided, too, for the laying up of vessels the bottoms of which have become foul.

Such advantages as are set forth in the paper and, to some extent repeated here, must be weighed, however, against the large investment that would have to be made to build a structure for the most part on a fine sand or mud bottom and nearly 2 miles in length, in water 60 ft., or more, in depth. Account must be taken, too, of the fact that boats plying on the Upper Bays and on the rivers would have to use locks when passing from San Pablo into San Francisco Bay or making the reverse trip.

The project cannot be fully discussed on its merits without further basic data than are now available. The advantages, as set forth, weighed against the disadvantages, would seem to justify a careful engineering study of the project.

H. H. WADSWORTH,* M. AM. SOC. C. E. (by letter.)—Any scheme which may be proposed for preventing the recurrence of so serious a condition as that which existed in Suisun Bay (constituting with San Pablo Bay an arm of San Francisco Bay) and in the lower reaches of the Sacramento and San Joaquin Rivers in the summer of 1920, is worthy of careful consideration. The fact that the scheme proposed by the author forms part of a so-called comprehensive plan for the alleviation of all the troubles of water users and flood sufferers, not only in the Great Valley of California, but throughout the greater part of the State (which plan has been the subject of much exploitation and propaganda during the past year and has been condemned by engineers generally as chimerical and impossible of attainment), causes people to look with skepticism on the parts of it outlined in the paper.

The first impression of one who has spent the greater part of sixteen years on studies of the reclamation, irrigation, and navigation problems and in observing

* San Francisco, Cal.

and experiencing construction difficulties in the region which would be affected by the proposed dam, is that the plan for such a dam also is visionary, because construction difficulties are too great to be overcome, at least within financing possibilities.

The facts that some commendation has been heard for this scheme by persons whose judgment has generally been sound and much respected, and that a bill has been introduced into the State Legislature appropriating $75 000 for an investigation and report on the feasibility, cost, general plan, and design for the construction of a dam at one of the alternative sites proposed by the author, have induced the writer to catalogue some of the difficulties which beset the carrying out of the proposed scheme and to analyze the resulting conditions sufficiently to form some idea of the character and necessary dimensions of the structure. This has not been done in sufficient detail to permit even a reasonably fair guess as to the cost of the structure, or to evaluate the differences between benefits and damages to lands in a great part of the Lower San Joaquin River delta.

As a result of these studies, the scheme does not seem to be quite as hopeless as at first thought, although it is not believed that the examples referred to by the author can properly be called supporting precedents, or that the cost can be kept below an amount several times as great as he thinks, as expressed by his statement that "it is probable that one or perhaps two yearly payments [of $3 000 000] would finance the construction of a dam equipped with a movable crest and multiple locks between Suisun Bay and the junction of the two rivers", or by another statement that the construction of "the Assuan Dam on the Nile, designed to pass a flood of 490 000 sec-ft. and store 800 000 acre-ft. for irrigation" was accomplished "at a cost far in excess of what it would require to accomplish" similar and other "objects in the Great Valley of California".

The cost of the original Assuan Dam was $11 900 000, or $14.95 per acre-ft. for the 795 000 acre-ft. capacity. The estimated cost of its enlargement in 1907 was $7 410 000, or $6.90 per acre-ft. for 1 070 000 acre-ft. increased capacity, thus making the final dam cost $19 310 000, or $10.35 per acre-ft. of its total capacity. These costs for water-storage capacity, per acre-foot, compare very favorably with proposed, and accomplished, storage reservoir projects in the United States, and are very much less than the amount which the author represents to be the cost of combating, by the release of stored water, the encroaching salinity and maintaining the required depth of water for navigation. That is to say, if the encroachment of salinity could be prevented by the release from storage of 3 000 sec-ft. for 100 days each year, by making available for that purpose 600 000 acre-ft. of storage capacity (without sacrificing a greater need of storage capacity for distributing, through the season, flood waters for irrigation), at a unit cost no greater than that at the Assuan Dam, the cost would be, in the writer's opinion, considerably less than that of the proposed scheme of constructing a dam at any of the suggested locations. Instead of being a moderate value, as stated by the author, $5 per acre-ft. per year, representing a capital expenditure of from $65 to $85 per acre-ft. of reservoir storage capacity, would be an excessive value of water for the purposes under consideration. There are, however, other features of the project for converting Suisun Bay, or both Suisun and San Pablo Bays, into a fresh-water lake, which may, perhaps, justify the cost.

The project for the control of floods on the Sacramento River now being carried out under the direction of the California Débris Commission with funds contributed by the Federal and State Governments provides channel capacity for 600 000 sec-ft. of water, with a mean water surface elevation at the mouth of the river of 7.0 ft., U. S. Engineer Department datum (which is 3.6 ft. below mean sea level and about 3.4 ft. below half-tide level at this place) and calls for levees of heights sufficient to protect the lands against tides 3 ft. above this mean flood elevation.

A tentative flood-control project for the San Joaquin River which serves as a temporary guide to the State and Federal authorities in passing on reclamation projects on the San Joaquin River is planned to provide for the discharge of that stream, at its mouth, of 260 000 sec-ft., with the same water surface elevation at the common mouth of the two rivers. The maximum simultaneous combined discharge of the two rivers is estimated at 750 000 sec-ft.

On account of the smaller cross-section of the Lower Sacramento River, as compared with the San Joaquin (even after being greatly enlarged in the execution of the flood-control project), the confinement of this river to practically a single channel, and the much greater volume of water which it must carry, the flood flow gradient is much steeper and the length of the channel in which tidal action (at high flood stages) is appreciable, is much less on the Sacramento than on the San Joaquin River and on the sloughs and canals between the islands of the San Joaquin delta. The estimated flood height at a point 15 miles above the mouth of the Sacramento River, for a discharge of 600 000 sec-ft., is at an elevation of 15.0 ft. This is at a point above which the formation is such that no great difficulty is experienced in building and maintaining levees at any reasonably desired elevation and section.

In the many channels of the San Joaquin delta, for a flood discharge of 260 000 sec-ft. (by the San Joaquin River), the estimated flood plane rises to an elevation of 15.0 ft., at a distance of from 30 to 40 miles above the mouth. The area of the delta, within the channels of which the computed flood heights (corresponding to the estimated maximum floods) are below an elevation of 15.0 ft. (U. S. Engineer Department datum), is about 300 sq. miles. Within this area are several hundred miles of channels bordered by levees. The computed mean tide elevations, for extreme flood conditions, in the channels of one-third this area, roughly speaking, are between 7.0 and 11.0 ft., with corresponding high-tide elevations of about 9.5 ft. and 11.5 ft., respectively.

In a large part of this smaller area of about 100 sq. miles, the underlying formation, consisting of tule roots, peat, and mud, is such that levees, sufficient for the protection of the land, are maintained with great difficulty. Some have failed at the occurrence of every severe flood. In some instances, high tides, even when flood conditions were far from extreme, have caused levee failures.

Bearing in mind the situation as described, it becomes evident that any structure which would throttle the flood discharge of the rivers so as to increase materially the up-river flood heights would meet with bitter opposition. To raise the low-water surface of the rivers in the vicinity of Sacramento and Stockton from 5 to 10 ft. above the highest tide, as suggested by the author, could be effected only by dams

many miles up stream from the mouths of the rivers. No relief from salt-water troubles, such as have already been experienced, would be effected by them.

The farther down the Bay a dam is built which will prevent the up-stream passage of salt water, the greater, of course, will be the area within which structures will be immune from the ravages of the teredo and other marine borers and the greater will be the volume of impounded fresh water made available for beneficial use; likewise, the greater will necessarily be the dimensions and capacity of the locks required for shipping, the greater will be the reduction of the tidal prism and the resulting scouring action which is the main factor for maintaining deep water across the bar outside the Golden Gate, and the greater difficulty there will probably be in securing the co-operation of the Federal Government.

Taking into consideration what is known of the formation underlying Suisun Bay and the adjacent marsh-lands, it is not at all "plainly seen" that the project for a dam at the head of Suisun Bay is of materially less "gigantic proportions" than one for a dam at the head of Carquinez Straits, from Army Point to Suisun Point. A dam at the lower end of San Pablo Bay, from San Pablo Point to San Pedro Point, would be 50% longer and would be in water having an average depth nearly 50% greater than one constructed at the head of Carquinez Straits.

At Collinsville (at the mouth of the Sacramento River), mean higher high water is at an elevation of 6.7 ft. (U. S. Engineer Department datum), and mean lower low water is at an elevation of 1.0 ft. The average water surface for two days during the flood of January, 1909, was at an elevation of 6.9 ft., and during this period the range of tide was from elevations of 5.1 to 9.7 ft.

In order to form some conception of the structure which would be required for functioning as contemplated by this scheme, at the three respective localities named, assume a combined discharge of the two rivers of 750 000 sec-ft. This is probably not far from the actual maximum discharge during the floods of March, 1907, and January, 1909. Assume, also, that the average water surface elevation at the mouth of the rivers for this discharge is increased, by reason of the dam, only 1.7 ft., or to an elevation of 8.6 ft. It is probable that the tidal range would be decreased somewhat, so that extreme high tide would perhaps be only about 1 ft. higher than in 1909 under tidal and meteorological conditions similar to those which then obtained.

The width and average depth of channel at the three suggested dam sites are approximately as follows:

	Width.	Average depth.
At Chipps Island	3 000 ft.	25 ft.
At Army Point	5 000 "	38 "
At San Pablo Point	7 500 "	50 "

The quantities involved, the volume of flowing water, and the dimensions of channels and of structures so far built transcend anything known in the way of measured flow or determination of coefficients to justify the use of a submerged weir formula for finding the depth below the surface to which the fixed, or immovable, part of the dam should be built so that the flood flow could pass it without causing a rise of the water surface greater than the allowable maximum previously assumed. It would seem that the minimum depth to the fixed structure should

be such that the cross-section area above it and the increased velocity due to the head of the allowable increase in mean elevation of water surface above the dam (assumed to be 1.7 ft. at extreme flood stage), would just provide for the extreme flood discharge.

Computations based on these data and assumptions show that the top of a fixed structure at Chipps Island would be at an elevation of — 10.6 ft. (U. S. Engineer Department datum), or about 20 ft. below the surface of the water at the highest tide. At the Army Point site, the top of the fixed structure would be at an elevation of — 5.8 ft., or about 14½ ft. below the surface of the water at the highest tide. At the San Pablo Point site the top of the fixed structure would be at an elevation of — 3.3 ft., or about 10½ ft. below the surface of the water at the highest tide. Then, in order to impound the water during low-water seasons at the elevation of the mean higher high water, there would have to be a movable crest 17 ft. high at Chipps Island, 12 ft. high at Army Point, or 9½ ft. high at San Pablo Point. A movable dam crest 12 ft. high, with its base 6 ft. below mean lower low water and 4½ ft. below extreme low water, such as would be required at the Army Point site with tidal currents as great as prevail there, would seem to the writer to be about the limit in height for reasonably satisfactory operation.

It seems to be the author's idea that the channel at the Chipps Island site could be widened and reliance placed on a large part of the flood flowing over the adjacent lowlands. The volume of water which could pass in comparatively thin sheets over lowlands compared with that flowing in an equal width of deep channel across which there is a narrow obstruction only (such as the submerged dam), is very small. To enlarge the channel by dredging so that the depth of the submerged dam might not considerably exceed that at the Army Point site would be of doubtful value, especially as the foundation conditions are probably no worse, to say the least, at the latter site.

The formation underlying the greater portion of the marsh-land adjacent to Suisun Bay and the lower reaches of the Sacramento and San Joaquin Rivers is soft mud to a great depth. This has been demonstrated by the struggle of the Southern Pacific Company for many years to maintain its railroad track for a distance of several miles, about 10 or 12 miles northeast of Chipps Island, by the Sacramento Short Line (Electric Railway) substituting pile structures for embankment along 3 or 4 miles of its line immediately north of Chipps Island, and by the excessive subsidence of levees built by the Government, by reclamation districts, and by private parties in this region. By reason of the conditions described and of the fact that the resulting alleviation of the trouble due to salt water would apply to a much smaller area, no further consideration will be given by the writer to the Chipps Island location.

Some fifteen or twenty years ago the Southern Pacific Company contemplated building a bridge across Carquinez Straits to replace the Port Costa-Benicia Ferry on its main transcontinental line. It is quite probable that one would have been built, if the Government had permitted a bridge at low level, with draw-spans. Test borings were made on a line across the Straits at Vallejo Junction and test piles were driven on a line from Army Point to Suisun Point. Through the courtesy of William Hood, M. Am. Soc. C. E., Chief Engineer of the Southern Pacific Company, the results of these tests have been made available to the writer.

Some of the data secured are shown on Fig. 6. The section at, or near, Vallejo Junction is of incidental interest in that it shows the depth to rock all across the channel, at a point about 7 miles west of the Army Point Dam site, and a depth of water of 100 ft.

On the Suisun Point-Army Point Section, Fig. 6, are shown the depths of water, the maximum being 50 ft., the depths to which test piles settled by their own weight, and the penetration of test piles and the settlement (at the penetrations shown) per blow of a 2 800-lb. hammer falling 20 ft. It will be noted that for about 2 500 ft. of the channel width, piles reached a penetration of from 96 to 106 ft. and that the settlement at these penetrations varied from 1½ to 3½ in. per blow. A pile foundation driven to these depths would probably safely carry a load of 16 tons per pile, or 2 tons per sq. ft. of the base of the structure, assuming piles to be driven on 2 ft. 10-in. centers.

Nothing is known of the subsurface conditions at the San Pablo Point site. The chart shows 75 ft. of water and a sand bottom. There are rock banks at each end and there is an outcropping rock island near by, which may be considered as indications that a firm foundation may be secured at a less depth below the bed of the channel than that at the other sites. For comparison with the conditions which govern the design of such a dam as that proposed, attention is called to certain features of the Assuan Dam, besides its cost and reservoir capacity to which reference has been made. It has a length of 6 200 ft., which was made necessary in order to pass the maximum possible flood discharge, and it was built on a rock foundation and in the open air.

The diagrams of suggested dam sections, Fig. 7, show the minimum sections, in practically the deepest water, for a continuous structure at each of the two sites, San Pablo Point and Army Point, under the hydrostatic head corresponding to the difference between mean higher high water and mean lower low water, which would not cause back-water elevations higher than those assumed to be allowable, and which would give an average base pressure of 2 tons per sq. ft. A bridge with a 20-ft. roadway is shown, which would serve both as a highway and as a working platform for raising and lowering the movable crest. In the case of the Army Point Section, Fig. 7, a line of sheet-piling is shown in the foundation, because without it the difference in head on the two sides might cause a flow of mud or a blowout under the dam.

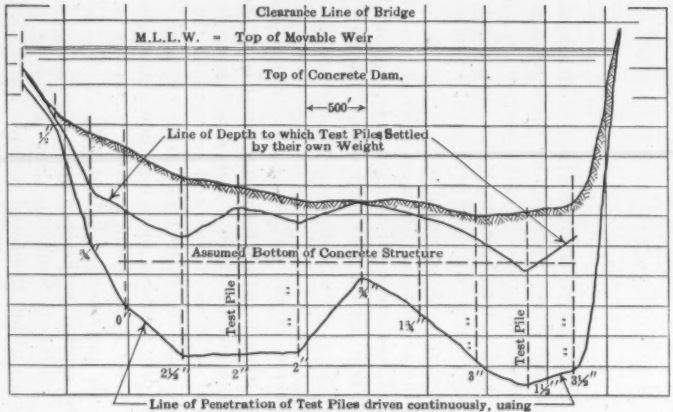
The structures suggested by these cross-sections would consist of an outer shell of reinforced concrete of any desired length for convenience in handling, and of such height that when in position the top would be at the elevation of the top of the fixed part of the dam. There would be a floor above the bottom of the shell, in which there would be an air-lock and below which the finishing excavation and concrete placing could be done under compressed air. Around the top there would be a coffer-dam reaching above high tide so that after being floated into position, sunk, the water pumped out, and the concreting done around the piles and under the floor, the remainder of the concrete, or other filling, could be placed and the movable crest erected in the open air.

It might be possible to build this dam by sinking open caissons, driving and cutting off the piles within them, and then placing the bulk, or a considerable part, of the concrete in the water; after which the water could be pumped out and the

CROSS-SECTION OF SUISUN BAY

FROM SUISUN POINT TO ARMY POINT.

Showing Elevations of Principal Lines of Proposed Dam, and some Data from Pile-Driving Tests made by Southern Pacific Co.



CROSS-SECTION OF CARQUINEZ STRAITS

NEAR VALLEJO JUNCTION.

Showing Results of Test Borings made by Southern Pacific Co.

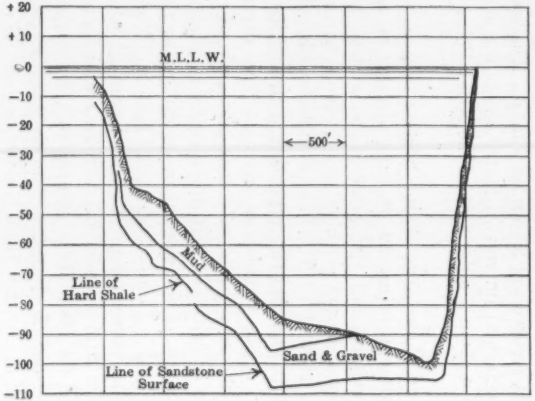


FIG. 6.

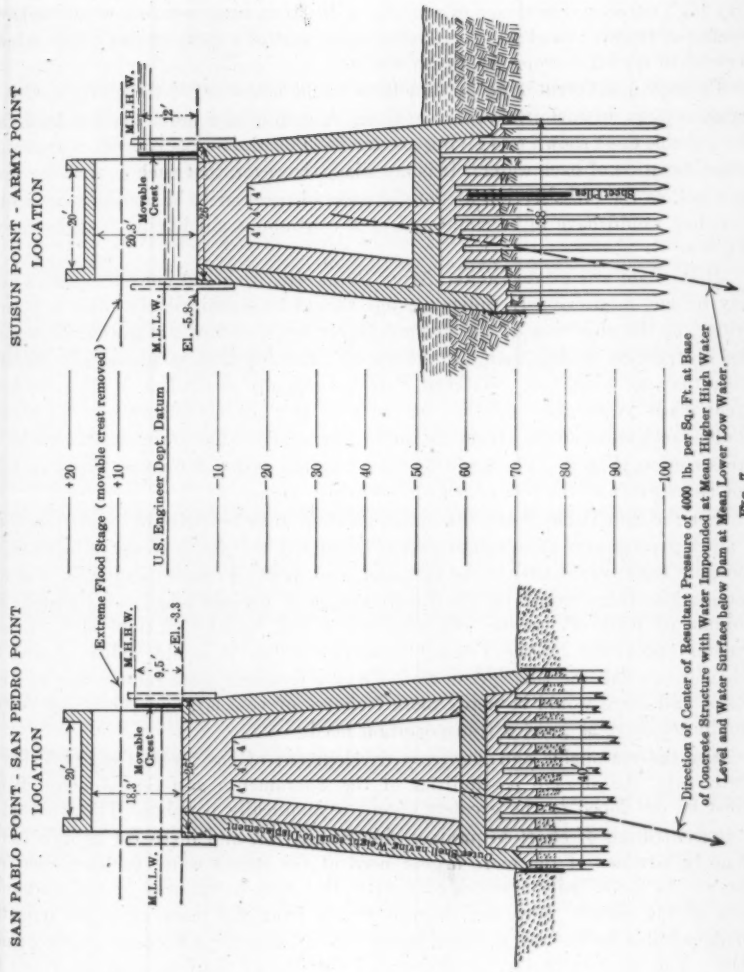

PROPOSED DAM SECTIONS
SAN PABLO POINT - SAN PEDRO POINT
LOCATION
SUISUN POINT - ARMY POINT
LOCATION
Extreme Flood Stage (movable crest removed)
U.S. Engineer Dept. Datum
M.H.H.W.
M.L.L.W.
Movable Crest
El. -3.3
El. -5.8
Direction of Center of Resultant Pressure of 4000 lb. per Sq. Ft. at Base of Concrete Structure with Water Impounded at Mean Higher High Water Level and Water Surface below Dam at Mean Lower Low Water.

FIG. 7.

structure completed. It has been conclusively demonstrated that to resist the destructive action of sea water, reinforced concrete must be of the greatest possible density, particularly between the elevations of high and low water. Although much (perhaps all) of this destructive action is due to rusting of the embedded steel reinforcement, other elements are present in this case—a channel subject to very high current velocities and a depth of 70 ft. of water—which would make the placing of freshly mixed plain concrete for any part of a structure on which so much depends of doubtful propriety, to say the least.

Perhaps a different type of structure would better serve the purpose, that is, separate piers, with shutters between them, extending to a much greater depth than the movable crest shown and giving greater flood-channel capacity with a consequent lesser elevation of back water. It is not likely, however, that such a structure would cost less, as the cross-section of the piers would necessarily be considerably greater and they would have to be connected by a continuous sill having footings for the shutters.

If the dam was located at the Army Point site, it should not only carry a highway bridge, as previously indicated, but should be combined also with a railroad bridge, as the objection of the Federal Government and of navigation interests to the obstruction would then be overcome. The location is practically identical with that on which the Southern Pacific Company desired to build a bridge a great many years ago, and the need of a bridge has not decreased. To carry a double-track railroad the structure would necessarily differ from that shown by the cross-section, Fig. 7. The additional cost of the railroad feature would probably not be as great as the cost of a railroad bridge alone.

For the San Pablo Point site, although there may be claimed the advantages of a much greater area of salt marsh-lands reclaimed, less depth of water in which the movable crest would have to be operated, and, possibly, more likelihood of a rock foundation, there would be the disadvantages of greater length of structure and volume of material, greater depth of water and volume of tidal flow, probably greater opposition by the Federal Government (as it would reduce during the summer and fall the tidal prism of the Bay very much more than a dam at Army Point and there would be many more lockages of vessels), and the bridge feature would not serve as great a transportation need.

For the reasonable and necessary development of the Great Central Valley of California, the storage possibilities of the Sacramento and San Joaquin Rivers must be developed to the greatest extent economically possible. There should be a concentration of effort to produce this result. If, and in so far as, this can be done in advance of the more urgent need of the stored water for diversion from the rivers, the salinity troubles of Suisun Bay may be relieved by the augmented flow of the rivers. Also, the seepage return from the more extensive irrigation made possible by the use of stored water will aid largely in keeping up the summer flow. The more extensive, or intensive, irrigation of the San Joaquin Valley has already increased materially the low-water flow over what it was a few years ago.

A careful investigation of the proposition of converting Suisun Bay into a fresh-water lake should undoubtedly be made. The cost of such an undertaking, however, would probably be so great that in order to accomplish it the co-operation of the Federal and State Governments, transportation interests, and property owners

would be required. The balance between benefits and damages to lands bordering on Suisun Bay and in the lower parts of the San Joaquin delta will be a delicate one.

The extent to which the up-river water users could be expected to co-operate, financially, would be measured by the cost to them of creating storage in excess of their irrigation needs so that water could be released to augment the low-water flow of the river to a degree such that they would not be liable for damages resulting from reduced discharge. If a temporary injunction which has been granted against certain up-river water users should be made permanent, it is evident from what is now known of reservoir possibilities in this State, that in determining the interests of the up-river irrigators in the Suisun Bay-Fresh-Water Lake Project, consideration will have to be given to mountain storage possibilities considerably beyond any which have heretofore been considererd to be economically feasible.

FRED. H. TIBBETTS,* M. AM. SOC. C. E. (by letter).—Some of the author's suggested assumptions may possibly lead to erroneous conceptions. It is suggested, for example, that water delivered from reservoirs for irrigation purposes "represents the moderate value of $5 per acre-ft." The writer believes this to be from perhaps four to ten times the current value of water for such purposes in this locality. The bulk of irrigation water used in 1920, in the Sacramento Valley, was for rice irrigation, and when delivered to the irrigators at the fields cost from about $1 to $2 per acre-ft., most of this cost having been for pumping and distribution. Capitalized, $5 per acre-ft. would represent an investment of from $60 to $100 per acre-ft. of storage. It is common knowledge that there are a number of large reservoir sites in the mountains at which storage can be obtained at a much smaller cost. If water was stored at the upper instead of the lower end of the river system, its use for irrigation would result in large quantities of "return water", naturally increasing the low-water flow of the stream.

In connection with the Antioch litigation, the maximum flow estimated by experts as necessary in the Sacramento River to prevent the encroachment of salt water in the delta, could readily be obtained by mountain storage at a cost which the writer believes would prove far less than the storage proposed in Suisun Bay. There are other factors to be considered in either case. It should be possible, to some extent, at least, to mitigate flood conditions by the large mountain storage suggested.

The assumption that "it is required to augment the natural flow of the Sacramento and San Joaquin Rivers by 3 000 sec-ft. for 100 days each year", necessitating "the retention of 600 000 acre-ft. in storage reservoirs, or else the deprivation of the irrigation interests of this amount", leads to confused ideas regarding irrigation requirements. The maximum amount of land which can be developed with certainty under irrigation, depends, under present conditions, on the unregulated low-water flow of the Sacramento River. The low-water flow varies greatly and is much affected by a series of years of sub-normal rainfall such as the last three and last five, and even the last ten, years. The unregulated low-water flow of the river which at present should be the governing factor in the extent of the irrigated area, could be increased more than 3 000 sec-ft., or

* San Francisco, Cal.

could practically be doubled by mountain storage at a cost which might easily prove only a small fraction of that of any one of the dams proposed by the author.

Serious consideration is now being given by Government engineers to the canalization of the Lower Sacramento River by means of movable dams. These dams could be located so that they would not only furnish slack-water navigation, but would materially raise the low-water plane of the river, decreasing the irrigation expenditures for pumping water over the river banks, and raising the water level in the Harbor of Sacramento, this latter being one of the advantages suggested by the author, although it is not clear how this could be accomplished by dams at any of the locations proposed. It seems probable that the lower dam for slack-water navigation could be placed far enough up the river so as to be on solid foundations, and, of course, at a cost not in any way comparable with that of the lower dams proposed by the author. From the site of such a lower slack-water dam, all the fresh-water flow of the Sacramento River reaching this point could be distributed for irrigation through the delta of the Sacramento and San Joaquin Rivers, utilizing chiefly the network of narrow winding sloughs, at present of little or no advantage, for conveying the flood-water.

Others have called attention to the exceedingly poor foundation conditions existing in the delta region and in Suisun Bay. The writer has supervised dredging operations in Suisun Bay, during which a large clam-shell bucket brought up material from a point 120 ft. below low-water surface; this material was a brown, fibrous peat with a strong odor of hydrogen sulphide, indicating vegetable decomposition still actively under way at this great depth. Under these conditions, the surface material will not stand the weight of an earth levee more than 8 to 10 ft. high, or probably not more than $\frac{1}{2}$ ton per sq. ft. The writer knows of no precedent and sees none in the list suggested by the author for any structure in any way comparable to those proposed, on such poor foundations.

A realization of these same foundation conditions is also very important in the discussion of any project which would increase, ever so little, the high-water flood plane in the 300 000 acres, or more, of highly developed and valuable delta lands, protected by levees which rest on such poor foundations that any material increases in their height and weight are impractical.

In conclusion, and referring to the author's final remark that "the principal purpose of this paper will have been achieved if the necessity for a regulating dam with locks near the mouth of the combined rivers has been established", the writer does not believe that this has been established; he does believe, however, that the author deserves the greatest credit for his courage in advancing a serious discussion of a project of such uncertainty and such magnitude. The writer also believes that the paper will serve even a far greater purpose if it helps to establish the necessity for the utilization of more of the Sacramento River water and for a comprehensive investigation of all the possibilities for increasing such use, including the solution which the author suggests and which may ultimately be found to be the best.

C. S. Jarvis,* M. Am. Soc. C. E. (by letter).—The writer is impressed with the valuable points and the tolerant views expressed toward a project of such magnitude, involving so many complications, uncertainties, and interests, and

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supported at present by such meager data, especially on foundation conditions at the various dam sites proposed.

The first sub-division of the subject suggested by Mr. Tibbetts, that of diversion of water from other water-sheds, was not contemplated in this paper; the second method was considered to be advancing at a satisfactory rate, but even when developed to the fullest extent there would be surplus water of these rivers which could not be retained in mountain reservoirs, and should be made to displace the salt water of the Upper Bays, as noted in the final paragraph of the Appendix. The canalization of the two rivers below Stockton and Sacramento and the storage of fresh water in the lower courses and Upper Bays were the main proposals of the paper, these phases having been generally neglected in discussion and development.

The value of $5 per acre-ft., which has been challenged by Mr. Tibbetts, was arrived at after considerable study of the question, and the quoted value was approved by many of the leading engineers and practical irrigation managers in the delta region. The actual maintenance cost of delivering water to the rice fields is not a proper comparison; but after the temporary injunction was rendered in 1920, preventing further diversion of river water, it is not improbable that the farmers adversely affected would gladly have paid a higher rate rather than see their growing crops deteriorate. As long as there is more water than all claimants can use, the value is apt to be expressed in terms of actual operating cost; but when a stream is over-appropriated, the water right itself where irrigation is required may approach or even exceed the price of the land served. It is difficult to see how the estimated value of $5 per acre-ft. can be from four to ten times the current value of water for such purposes, as mentioned by Mr. Tibbetts.

It is not conceded that "solid foundations" exist at any point in the course of the lower river; nor that the network of sloughs could be utilized below such a dam for distribution of irrigation water, as suggested by Mr. Tibbetts; neither is it conceded that the quagmire conditions of the Suisun marshes prevail at the Chipp's Island dam site. The piling supporting the ferry slips for the electric railway near this proposed site, and the trestle work for the same railway northward, together with the swift current that prevails at times in this channel, preventing the deposit of the finer sediment, are proofs to the contrary; but positive data derived from borings are needed at each site considered.

One of the most valuable contributions to the present data on this problem, if susceptible of proof, is the statement by Mr. Tibbetts that the flow of the Sacramento River "could be increased more than 3 000 sec-ft., or could practically be doubled by mountain storage at a cost which might easily prove only a small fraction of that on any one of the dams proposed by the author." If this claim can be established, it should take precedence over all other proposed solutions; but grave doubts are entertained as to the validity of such a claim.

If serious consideration is now being given by Government engineers to the canalization of the Lower Sacramento River by means of movable dams, they must be convinced that the navigation interests alone can justify the construction. Then, if a single structure placed below the junction of the two rivers raises the water plane to both Sacramento and Stockton, thus furnishing a deeper channel for both rivers, and serves irrigation and community water supply projects in addition, the single structure would be favored even at increased cost. From data

obtained during August, 1920, and recorded in the paper, the slope of the water surface with the least flow ever recorded was approximately 3 ft. from Sacramento to the head of Suisun Bay, 61 miles distant. Therefore, a dam at the junction of the two rivers, or at Army Point, or at Point San Pablo or Point Richmond, maintaining the water surface at mean high-water level, would increase the effective channel depth at Sacramento by 3 ft. or more.

If it is desired to elevate and maintain the water surface at a higher level along the river channels in order to increase the navigable depths, the dam site at Chipp's Island, together with retaining levees each side of the rivers for several miles up stream, would be required, or separate dams farther up stream.

Mr. Grunsky has given an excellent summing up of advantages and disadvantages that would accrue from the freshening of the Upper Bays, the cost of the structures and the delays at the locks for shipping being the principal objections. If the benefits due to freshening the Upper Bays should include the replacement cost of timber structures now being destroyed by marine borers, the value to arid and swamp land that could be reclaimed and made productive, the maintenance of a greater depth of channel for shipping by eliminating the low phase of the tide, and the influence on manufacturing and commercial activity by reason of such a distribution of fresh water from Point Richmond to Vallejo, Benicia, and Fairfield, and extending northward to Petaluma and Napa, it would be difficult to estimate the actual value of the proposed project. Perhaps the most serious objection to such a structure would arise on account of the delay in the passage of warships; and for this reason one of the upper sites, such as Army Point, might finally be chosen. The data included in Mr. Wadsworth's discussion, showing the results of borings for the proposed railroad bridge at this point, are not unfavorable.

It is noted with considerable surprise that Mr. Wadsworth states "that the scheme proposed by the author forms part of a so-called comprehensive plan for the alleviation of all the troubles of water users and flood sufferers, not only in the Great Valley of California, but throughout the greater part of the State"; and, in reply, the writer must claim that there is no relation unless by adoption on the part of proponents of the larger plan. By the same process of reasoning, every important suggestion by any writer on flood control or water supply problems in California is subject to the same charge of dealing with chimerical and impossible projects because of their being included in the so-called comprehensive plan. Probably more errors are committed under the name of engineering by reason of a too restricted view than by reason of the comprehensive plan.

Mr. Wadsworth's broad experience in the problems of river control in the region under consideration enables him to speak with assurance and contribute valuable data on the vital problems. However, it has not yet been discovered that 600 000 acre-ft. of water for maintaining a flow of 3 000 sec-ft. for 100 days each year can be stored at the same rate as that obtained at the Assuan Dam; nor that the cost of storing 1 acre-ft. of water in the mountains represents either the cost of delivery of the water at the land, or the value of the water during an extremely dry year. The losses in storage, in transit, and in distribution, and the cost of administration and operation of such enterprises, may readily be overlooked, as in this discussion, but they are large factors nevertheless.

The designs and discussions of types of structures and various locations are enlightening, and serve to emphasize the need of comprehensive investigations before the project is either approved or definitely abandoned.

The description of the maximum height of flood plane that obtains among the delta levees merely confirms the suggestion that added depth of navigable channel can best be obtained by elevating the river surface up stream from the junction during the season of low water.

Apparently, all the discussors assume that unstable foundation conditions obtain at each of the proposed dam sites. Some engineers who have studied the problem have favored a rock-fill dam with a concrete core for the greater part of the structure, the flood-gates to operate over apertures in the masonry core, the locks to be located at the ends of the structure. Taking into account the limited variation of water pressure on the two sides of the proposed structure, it would not be such an expensive project as many have assumed, but more basic data and designs will be required before a reliable estimate can be ventured.

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Paper No. 1473

PARABOLIC WEIRS

By F. W. Greve,* Esq.

With Discussion by Messrs. B. F. Groat, E. G. Walker, R. L. Sackett, John H. Gregory, and F. W. Greve.

Synopsis

This paper describes the parabolic weir, which was developed to:

1.—Facilitate the use of a comparatively simple and accurate recorder for measuring discharge.

2.—Develop a notch that would give a wider range in head for a given discharge than the rectangular opening.

3.—Produce a formula easy of computation.

A series of experiments on seven parabolic weirs was made in the Hydraulic Laboratory of Purdue University, the investigation covering a period of 9½ months. The notches were cut in ⅛-in. brass plates with the exception of one, for which the thickness was $\frac{1}{16}$ in. Three of the weirs were calibrated with the edge of the opening both beveled and unbeveled, while the remainder were tested for the former condition only.

The weirs were placed at the ends of suitable channels from which the discharge passed into a calibrated weighing tank with a capacity of 20 tons. Time was noted on a stop-watch electrically operated by the weighing scales. The elevation head on the weir was measured by a piezometer and at least one hook-gauge. A second hook-gauge was used in several of the tests to ascertain what drop, if any, occurred in the water surface within the channel of approach. False bottom and sides were used in the channel to produce a wide range in the velocity of approach.

Calibration tests were also conducted on a recorder that indicated the quantity of water passing through the weir for a given period of time. The device was operated by a float, quantities being indicated on both a chart and a counter. The error in the meter was less than 1 per cent.

The results of the investigation indicated that:

1.—The parabolic weir is one of the best types developed and is adaptable to a wide range of conditions.

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2.—The actual rate of discharge can be computed by the formulas:

$$q = kh^2,$$

and

$$q = C\frac{\pi}{4}\sqrt{2g}\sqrt{2p}\,h^2$$

where q is the actual rate of discharge, in cubic feet per second; h is the elevation head, in feet; k is a constant for any given weir with beveled notch; C is the coefficient of discharge; $\pi = 3.1416$; $g = 32.16$ ft. per sec. per sec.; and p is a constant in the equation of a parabola, $x^2 = 2py$.

3.—The parabolic weir is adaptable to a simple and accurate meter for measuring the discharge.

4.—The values of the constant, k, can be accurately predicted for any given value of p.

The parabolic weir is the invention of Mr. C. Lauritson, of the Ohio Body and Blower Company, Cleveland, Ohio. Calibration tests made on one of the weirs for the Company, after consultation with the writer regarding the proper design of the notch, was the incentive for the more thorough investigation which forms the basis of the paper. The writer desires to thank Mr. Lauritson for his able assistance and suggestions; also Prof. G. A. Young, Head of the School of Mechanical Engineering, Purdue University, Mr. R. R. Martin, Research Assistant in Hydraulics, Purdue University, and the Ohio Body and Blower Company.

A weir is a notch in the side of an open vessel or in a structure placed across a stream and is used to measure the quantity of flow. The rate of discharge from a weir depends upon the elevation of the water surface above the lowest point of the notch (elevation head) and upon the head due to the velocity of the stream (velocity head). Both these heads are measured at a point up-stream from the weir where the normal flow is uninfluenced by the change in cross-section of the stream as it passes through the notch. The best results are obtained when the velocity head is a negligible factor in computing the total.

The origin of the weir is unknown, and previous to the Nineteenth Century few data were available concerning the relation of actual to theoretical rate of discharge. A number of thorough calibration tests were conducted in France between 1825 and 1850 by Castel, Eytelwein, Poncelot, Boileau and others, but little research work was done in the United States until 1848, when the late James B. Francis, Past-President, Am. Soc. C. E., began his famous experiments at Lowell, Mass. The results of Francis' monumental work have been accepted as standard in this country. European practice is largely dependent upon the investigations of Bazin, who commenced his experiments in France in 1886. Research work has been largely confined to the rectangular notch and has been primarily for the purpose of establishing the relation of actual to theoretical rate of discharge, secondary consideration being given to the effect of the velocity of approach. With this type of weir, the rate of discharge varies as the three-halves power of the head, and the coefficient of discharge is a variable decreasing with increase of head.

The width of the stream through a rectangular weir remains constant, and consequently the cross-sectional area varies directly with the head or rate of discharge. A small error in reading the elevation head for low rates of flow

will produce a relatively large discrepancy in the results. The source of the greatest error in the use of weirs lies in the measurement of the head. The writer has observed that two investigators will frequently record slightly different readings for a given setting of a hook-gauge.

The triangular notch in which the rate of discharge varies as the five-halves power of the head is shaped so that the ratio of head to cross-sectional area of the stream through the opening decreases with increase of discharge. Thus, the head is larger in proportion to the area for minimum than for maximum rate of flow, and a mistake in reading a gauge produces a smaller discrepancy in the results than would be the case with a rectangular weir under like conditions as to capacity. The coefficient of discharge for a triangular notch is generally assumed to be constant for a given opening, although this assumption is probably not strictly true, especially for heads of less than 0.1 ft. The trapezoidal weir, particularly of the Cippoletti type in which the side slope is 1 to 4, is frequently used instead of the rectangular notch because of less variation in the coefficient of discharge and because the shape of the opening makes unnecessary any correction for end contractions. Other types of weirs have been designed for special purposes.

The design of the parabolic weir was undertaken with the following purposes in mind:

1.—To facilitate the use of a comparatively simple device for recording the discharge.

2.—To have the width of the stream through the notch increase with the head.

3.—To produce a formula easy of computation.

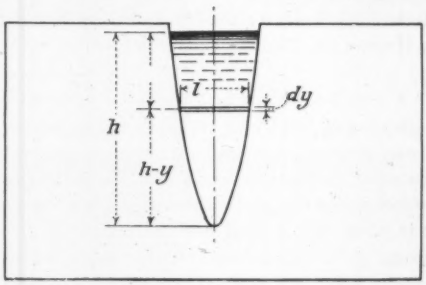

FIG. 1.

The equation for computing the rate of discharge through a parabolic weir may be derived by two methods.

Any horizontal strip of water of infinitesimal thickness, dy, is considered as an orifice, the theoretical rate of discharge through which is dQ and is equal to the product of the area and the velocity due to the head, y, on the center of gravity of the strip under discussion. The equation of a parabola is:

$$x^2 = 2\,p\,y = 4\,a\,y.$$

METHOD 1.

If V is the theoretical velocity, in feet per second,

$$V = \sqrt{2\,g\,y} \qquad (1)$$

and if Q is the theoretical rate of discharge, in cubic feet per second,

$$d\,Q = l\,(d\,y)\,\sqrt{2\,g\,y} \qquad (2)$$

From the equation of a parabola,

$$l = 2\,\sqrt{2\,p\,(h - y)} \qquad (3)$$

Substituting Equation (3) in Equation (2),

$$d\,Q = 2\,\sqrt{2\,p\,(h - y)}\,(d\,y)\,\sqrt{2\,g\,y} \qquad (4)$$

The total theoretical rate of discharge is,

$$Q = 2\sqrt{2g}\sqrt{2p}\int_0^h \sqrt{y(h-y)}\,(dy) \quad \ldots\ldots (5)$$

Taking the geometric mean between h and $(h-y)$,

$$x^2 = y(h-y) \quad \ldots\ldots (6)$$

Substituting Equation (6) in Equation (5),

$$Q = 2\sqrt{2g}\sqrt{2p}\int_0^h x\,(dy) \quad \ldots\ldots (7)$$

Now, $\int_0^h x\,(dy)$ is the area of a semi-circle divided by h as diameter, and Equation (7) becomes

$$Q = \frac{\pi}{4}\sqrt{2g}\sqrt{2p}\,h^2 \quad \ldots\ldots (8)$$

Method 2.

Using the foregoing notation

$$dQ = 2\sqrt{4ay}\sqrt{2g}\sqrt{(h-y)}\,dy \quad \ldots\ldots (9)$$

$$dQ = 4\sqrt{2g}\sqrt{a(hy-y^2)}\,dy \quad \ldots\ldots (10)$$

Integrating:

$$\int_0^h \sqrt{hy-y^2}\,dy = \frac{2y-h}{4}\sqrt{hy-y^2} + \frac{1}{2m}\int_0^h \frac{dy}{\sqrt{hy-y^2}} \quad \ldots (11)$$

where

$$m = \frac{-4}{-2} = 2 \quad \ldots\ldots (12)$$

Then,

$$dQ = \frac{2y-h}{4}\sqrt{hy-y^2} + \frac{h^2}{8}\int_0^h \frac{dy}{\sqrt{hy-y^2}} \quad \ldots\ldots (13)$$

$$\int_0^h \sqrt{hy-y^2}\,dy = \left[\frac{2y-h}{4}\sqrt{hy-y^2} + \frac{h^2}{8}\left(\sin^{-1}\left\{\frac{2y-h}{h}\right\}\right)\right]_0^h \quad \ldots (14)$$

$$\int_0^h \sqrt{hy-y^2}\,dy = \frac{h^2}{8}\left[\frac{\pi}{2} + \frac{\pi}{2}\right] = \frac{\pi h^2}{8} \quad \ldots\ldots (15)$$

$$Q = 4\sqrt{2ag}\,\frac{\pi h^2}{8} = \frac{\pi}{2}\sqrt{2ag}\,h^2 \quad \ldots\ldots (16)$$

If q is the actual rate of discharge, in cubic feet per second, and C is the coefficient of discharge,

$$q = CQ = C\frac{\pi}{2}\sqrt{2g}\sqrt{a}\,h^2 \quad \ldots\ldots (17)$$

and,

$$q = kh^2 \quad \ldots\ldots (18)$$

$$k = C\frac{\pi}{2}\sqrt{2g}\sqrt{a} \quad \ldots\ldots (19)$$

Description of Parabolic Weirs.

The parabolic weirs illustrated in Fig. 2 are designated according to the value of p in the equation of a parabola, $x^2 = 2py$. The values of p for the seven notches are 0.1000, 0.1118, 0.3000, 0.4465, 0.5000, 1.250 and 2.000 in., respectively. These figures will be used to designate the notches, which were cut in $\frac{1}{8}$-in. brass plates,

with the exception of No. 0.1000, which was formed from a $\frac{1}{16}$-in. plate. Templates were furnished to the shop as guides for cutting the weirs. Nos. 0.1000, 0.1118, and 0.3000 were made in the Purdue University shops; the others were formed at the plant of the Ohio Body and Blower Company.

Weirs Nos. 0.1000, 0.1118, and 2.000 were calibrated both before and after the edges of the notches were beveled. The others were tested for the condition of beveled notch only. The purpose of the experiments with unbeveled openings was to determine the necessity of a sharp-edged weir. The bevel was made at an angle of 30° to the plane of the weir, allowing a thickness of metal of $\frac{1}{32}$ in.

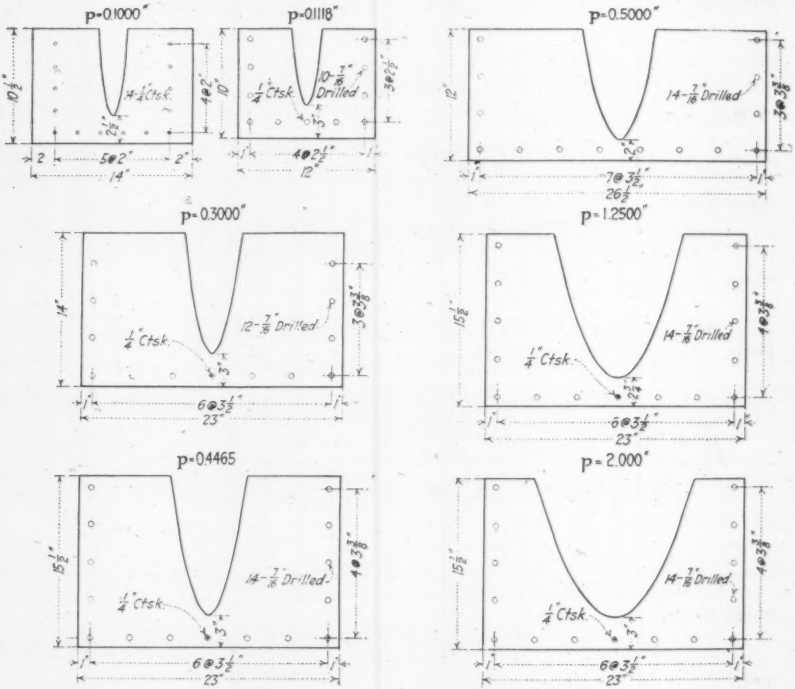

FIG. 2.

in contact with the water. The weirs were fastened to a heavy wooden frame set across the end of the channel. Weir No. 0.1118 was held to the frame by $\frac{1}{4}$-in. wood screws, the holes for which were countersunk, so that the heads of the screws were flush with the weir plate. The other weirs were secured by $\frac{3}{8}$-in. bolts, the heads of which projected into the channel of approach. A notch was cut in the wooden frame wider and deeper than the weir opening to insure an unobstructed passage for the issuing stream.

Description of Apparatus.

Weir No. 2.000 was the first to be tested and was placed at the end of the concrete channel, 5 ft. wide, the crest elevation being 4 ft. 8 in. The elevation head was measured by both a piezometer and a hook-gauge, Fig. 3. Readings could be noted to 0.001 ft. by means of verniers. The piezometer was connected to the channel by a ½-in. pipe, the entrance to which was flush with the interior face of the wall, within 6 in. of the bottom and 9 ft. 8 in. up stream from the weir. The hook-gauge was suspended from an overhead girder at the same distance from the weir as the piezometer. A calibrated stop-watch was used to determine the time. The flow was provided by either a 2-in., 60-gal. per min., triplex pump or a 6-in., 1 800-gal. per min., centrifugal unit, according to the quantity desired. The water was pumped through a 6-in. pipe, provided with a valve, to the concrete channel at a point 40 ft. up stream from the weir. The flow in the channel was over an 8-ft. suppressed rectangular weir and through two baffles. Zero head was noted on the gauges when the water surface was level with the lowest point of the notch as ascertained with a spirit-level.

All weirs other than No. 2.000 were placed at the end of a wooden channel, Fig. 4, that was 7 ft. 9 in. by 3 ft. 10 in. in plan and 2 ft. deep. The channel was constructed of smooth cypress planks, 1½ in. thick, and properly caulked to prevent leakage. The cross-sectional area could be changed by adjusting the false sides and bottom. Water entered the channel close to the up-stream end through a 3-in. connection to the 6-in. main. To allow the water to enter the channel with the least possible disturbance, the lower portion of the inlet pipe was drilled full of ⅜-in. holes. Two baffles were used to provide a smooth water surface.

The elevation head was at first measured with a piezometer and a hook-gauge. Later, a second hook-gauge was used to detect any drop in the level of the water surface, the gauge readings being noted as nearly at the same time as possible. The piezometer connection was a ¼-in. pipe, its entrance set flush with the inside face of the channel at a distance of 18½ in. up stream from the weirs and $2\frac{7}{16}$ in. below the level of the crest. The crest is herein defined as the lowest point of the notch. The hook-gauges were attached to rigid supports and were placed midway between the sides of the channel. Hook-gauge No. 1 was 4 ft. 5½ in., and No. 2, 18½ in. up stream from the weirs. An electric light was provided for each gauge to aid in taking the readings. The discharge was weighed in a tank of 20 tons capacity.

Fig. 5 shows the two channels and the apparatus for measuring the discharge. At first, the gauge readings for zero head were noted with the aid of a spirit-level which was later discarded in favor of a point-gauge, made of ⅜-in. brass stock, and fastened to the floor of the channel. The upper portion of this gauge was pointed and could be adjusted to any desired height. A connection was also made in the side of the channel for a mechanism that would record the total quantity of water passing through the weir. Apparatus similar to that adopted in connection with the concrete channel was used to maintain and measure the flow.

An electric circuit was contrived to operate the stop-watch in order to secure absolute accuracy in measuring the time. The circuit consisted of several dry cells, a switch, an electric magnet, a metal disk and two contact points. One contact point was attached to the smaller end of the balance arm of the scale and

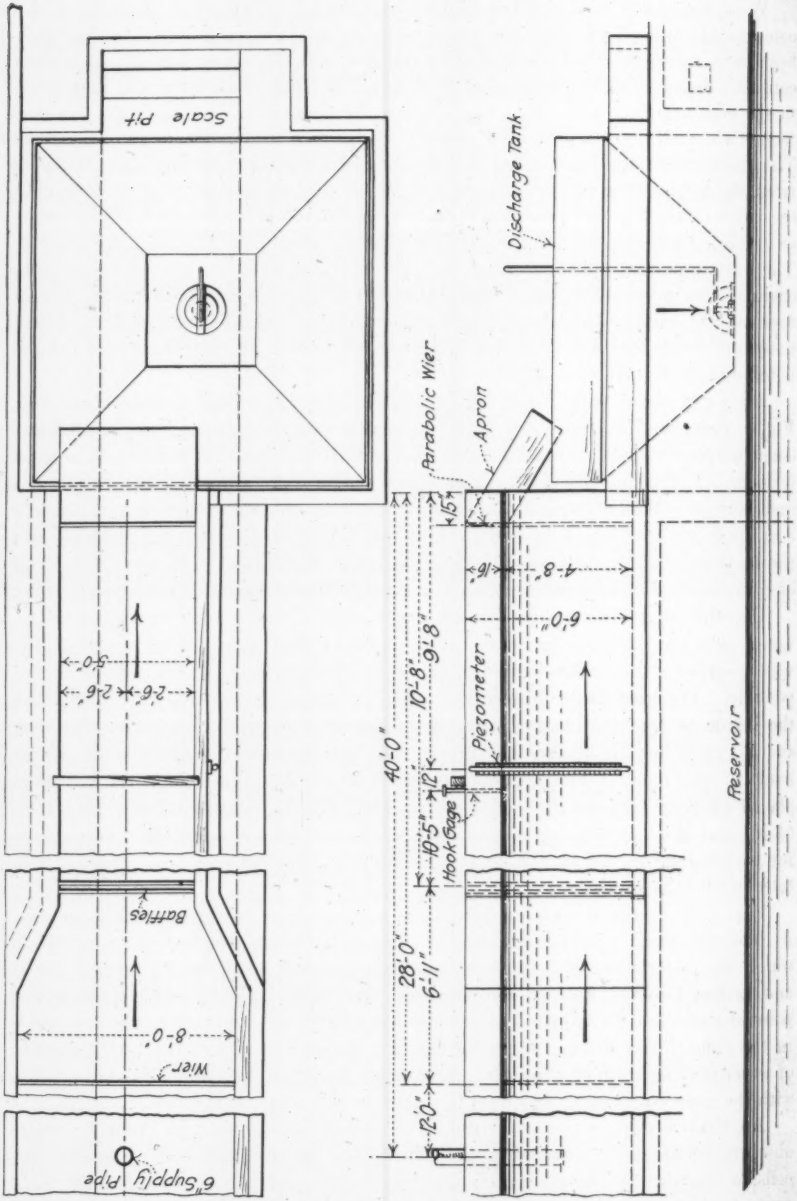
Scale Pit
Discharge Tank
Parabolic Weir
Apron
Piezometer
Hook Gage
Reservoir
Baffles
Weir
6" Supply Pipe
40'-0"
28'-0"
12'-0"
10'-8"
9'-8"
6'-11"
10'-5"
8'-0"
6'-0"
5'-0"
4'-8"
2'-6"
2'-6"

FIG. 3.

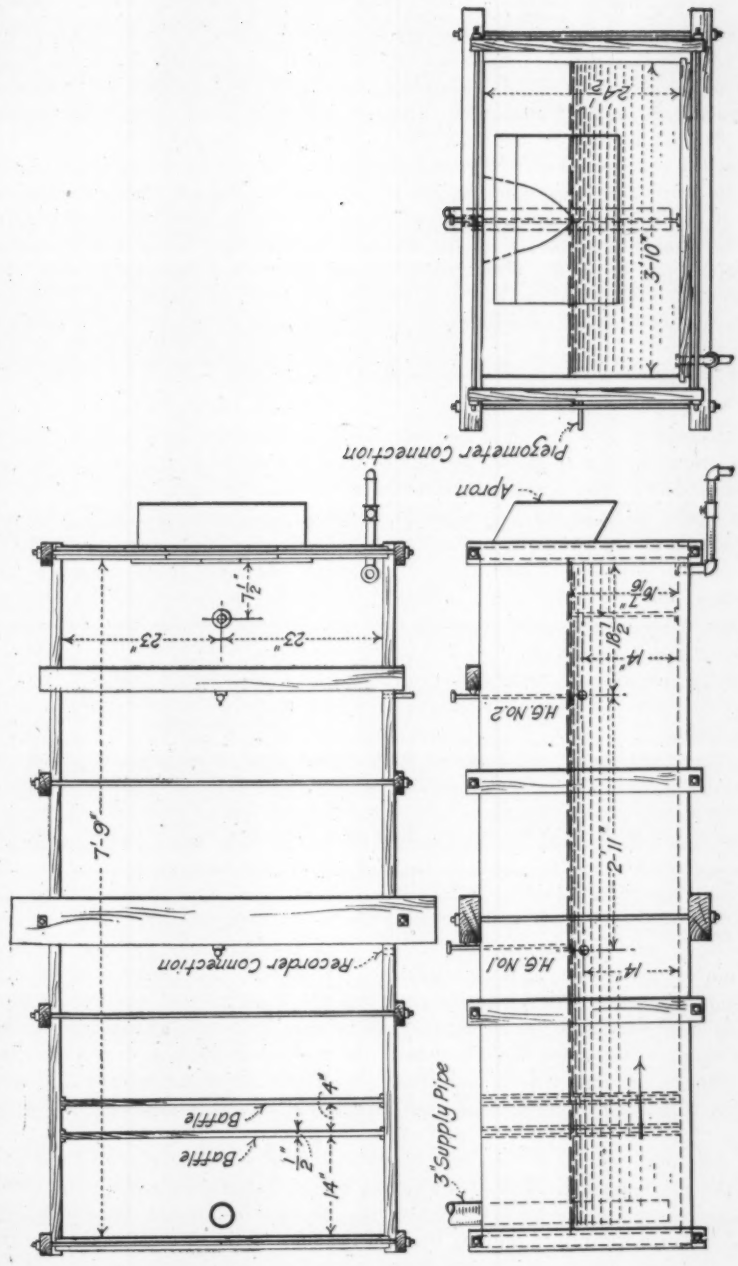
24"
3'-10"
Piezometer Connection
Apron
16 1/8"
18 1/2"
14"
H.G. No.2
2'-11"
14"
H.G. No.1
3" Supply Pipe
7 1/2"
23"
23"
7'-9"
Recorder Connection
Baffle
Baffle
4"
1/2"
14"

FIG. 4.

the other was fixed. The scale weights were adjusted so that the short end of the balance arm was raised and the valve in the discharge tank was closed. The switch was then thrown in and when the weight in the tank was sufficient to cause the movable contact point to be lowered, the circuit was completed and the magnet attracted the metal disk that was attached to a small lever. This lever was held in place by a hinge at one end. When the circuit was open, the lever and the attached disk were kept from contact with the magnet by a spring. The lever in its movement toward the magnet pressed on the stop-watch and the measurement of the time interval began. Opening the switch broke the circuit and the disk resumed its normal position. The scale weights were then set for a predetermined quantity in the discharge tank, and the switch was again closed. The watch was stopped when the circuit was closed a second time by the lowering of the short end of the balance arm.

Method of Conducting Tests.

Weir No. 2.000 was first placed in the large channel before being beveled. The gauge readings for zero head were noted when the water level in the channel was brought up flush with the bottom of the notch as indicated by a straight edge and a spirit-level. The pump was started, and the flow through the weir was wasted through a valve in the bottom of the weighing tank until the head had become steady as shown by the gauges. The waste valve was then closed and the time noted for a predetermined rate of discharge. At least three readings of the gauges, and frequently more, were made for each run or test, the length of the test being measured by the watch and determined by the time required to pass the quantity of water indicated by the change in weights. The usual practice throughout the investigation was to make two or more runs for each head. The weir was then removed, beveled, replaced, and tested under the same conditions as just set forth.

The other weirs were installed in the small channel and tested in the same manner as Weir No. 2.000. Weirs Nos. 0.1000, and 0.1118 were calibrated with the edges both beveled and unbeveled. For the first few tests, the position of zero head was found as indicated in the preceding paragraph. The point-gauge was then set up and the readings for zero head were obtained as follows:

The water level in the channel was brought to an approximate height $\frac{1}{16}$ in. below the bottom of the notch, and by means of a straight-edge and spirit-level, the tip of the point-gauge was set level with the weir crest. Water was slowly admitted to the channel until its surface was at the same elevation as the tip of the point-gauge, and the piezometer and the hook-gauges were read for this position of zero head. A discrepancy might arise if the point-gauge were set with the channel empty, for if the bottom of the channel should sag, due to the weight of the water admitted in adjusting the hook-gauges, the point-gauge would settle with the bottom of the channel and would no longer indicate the true zero head.

The inlet pipe as at first arranged was located in one corner of the channel at the up-stream end. This plan led to an uneven distribution of the water through the notch at high heads. The layout was changed before any tests were begun and the pipe was placed midway between the sides of the channel at the up-stream

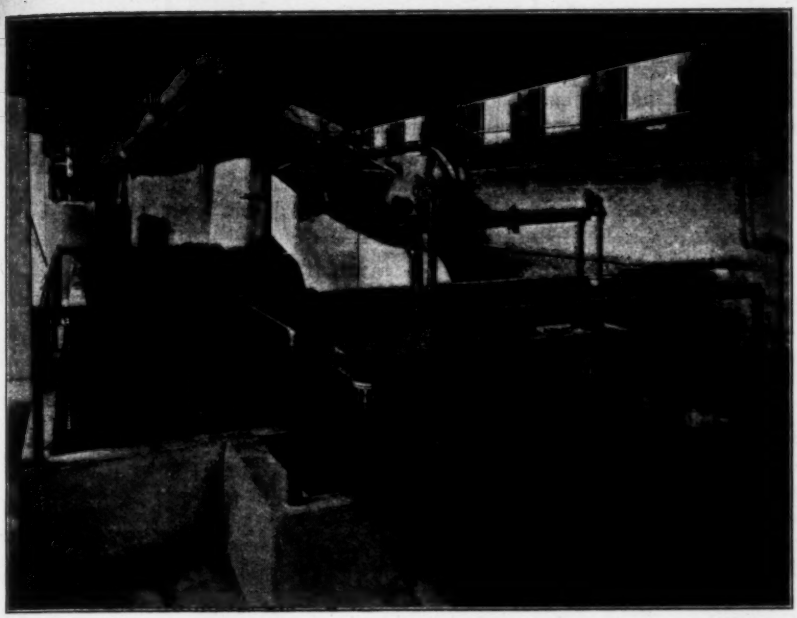

FIG. 5.—DISCHARGE TANK AND SCALES.

FIG. 6.—DISCHARGE FROM WEIR.

end, which arrangement proved to be satisfactory. False sides and bottom were provided to study the effect of change of cross-sectional area upon the rate of discharge for Weir No. 0.4465. The dimensions of the several channels are listed in Table 1.

TABLE 1.—Dimensions of Channels for Weir No. 0.4465.
b = Width of Channel; d = Crest Elevation.

Channel No.	b, in feet.	d, in feet.
1	4.0	1.37
2	4.0	0.25
3	4.0	0.00
4	2.0	0.25
5	1.0	0.25

The two hook-gauges were relied on for readings of the head when the width of the channel was less than 4 ft., the connection for the piezometer remaining undisturbed. The water surface in the 2-ft. and 1-ft. channels was agitated at high rates of discharge and, in consequence, the head had to be estimated in the last decimal place. When the water struck the weir plate in the 1-ft. channel at high velocities, it was first deflected downward and backward and then upward, giving rise to "boils" on the surface.

The constancy of the head is well illustrated in Fig. 6. This photograph required a long exposure, and yet the outline of the stream is as clear-cut as if taken with a snapshot. The stream itself was beautiful, giving the impression of cut glass. The surface of the water in the plane of the weir was curved downward, the degree of curvature increasing with the head and being greatest at a point midway between the sides of the notch. There was at all times a distinct line of demarcation between the nappe, or issuing stream, and the water in the channel. This line of demarcation took the form of two semi-circles of equal radii with the centers of curvature in or very close to the plane of the weir, the curvature being up stream. The nappe was of parabolic form until the point of maximum stream contraction was reached, after which it broadened out into a thin sheet, curved in outline and with the center of curvature at or very near the weir. The upper surface of the nappe was smooth at all times. The under surface was marked with what might be called "lines of flow" that crossed from side to side before reaching the point of maximum stream contraction. From that point on the nappe was a series of "ribs", being bound together on the upper surface by a thin sheet of smooth water. The nappe from the smallest weir at extremely low heads was twisted through an angle of 90° just prior to impact on the apron of the weir, giving rise to a thin sheet of water at right angles to the weir. It was noticed that suppression of the bottom contraction of the nappe began at heads ranging from 0.10 to 0.15 ft., irrespective of the size of the notch.

A mechanism, devised by Mr. Lauritson to record the total quantity of flow through a notch, was attached to the side of the channel at a point 4 ft. 5½ in. up stream from the weir. This mechanism was actuated by a float attached to a shaft by a rod. Fastened to the other end of the shaft was a pointer that was caused to move across the face of a dial as the shaft rotated due to the motion of

the float. The principle is simple. The pointer is set at the center of the dial for the position of zero head. The float will rise with increase of head, causing the pointer to move outward. The distance traveled by the pointer can be made proportional to that through which the float rises with the aid of a properly designed link. The cross-sectional area of the stream in the plane of the weir will vary as the square of the head which is also the square of the distance through which the float rises. The area of any circular disk with its center on that of the dial will vary as the square of the radius. The square of the head and the square of the radius are proportional to each other. Therefore, the area of any part of a circle described by the pointer is proportional to the area of the stream in the plane of the weir, both being dependent on the square of the head. Mr. Lauritson has also invented an automatic integrating attachment for the recorder which makes unnecessary the use of a planimeter on the chart. This latter attachment is somewhat novel in design in that areas are integrated directly. The rate of flow through the weir can be observed on the chart and the total amount can be read directly on the integrator.

The accuracy of both recorder and integrator is shown in Table 2. The tests were made with Weir No. 0.1118 in place, and the rate of discharge was varied several times during a test. The results indicated by the recorder are not as accurate as those shown by the integrator, because the former had to be obtained with a planimeter while the latter were read directly from the instrument.

TABLE 2.—Test of Recorder and Integrator.

Run No.	Weight, in pounds.	Recorder.		Integrator.	
		Pounds.	Accuracy.	Pounds.	Accuracy.
1	34 900	34 807	0.997	.....	.....
2	17 595	17 455	0.992	.....	.....
3	29 855	30 339	0.984	30 131	0.991
4	36 325	.....	.....	36 865	0.999
5	29 180	29 092	0.997	29 092	0.997
Average..	.....	.....	0.993	.....	0.996

Results of Calibration Tests.

The results of the calibration tests are summarized in Tables 3 and 4, and are graphically represented in Figs. 7 to 15, inclusive.

The slope of the straight line graphs was determined mathematically by the center of gravity method. Briefly, this method consists of first finding the center of gravity of all points. The points on either side of this center are then treated separately to locate their respective centers of gravity. If the ratio of the co-ordinates at the three centers is a constant, then the slope of the line is a constant and the graph is indeed a straight line. The plotted points on all the graphs lie close to the line, indicating a small departure from the average values for any one run. The results have been computed to three significant figures, irrespective of the decimal point. Runs in which the bottom contraction of the nappe was incomplete, as noted in Tables 3 and 4, have been omitted in the calculations.

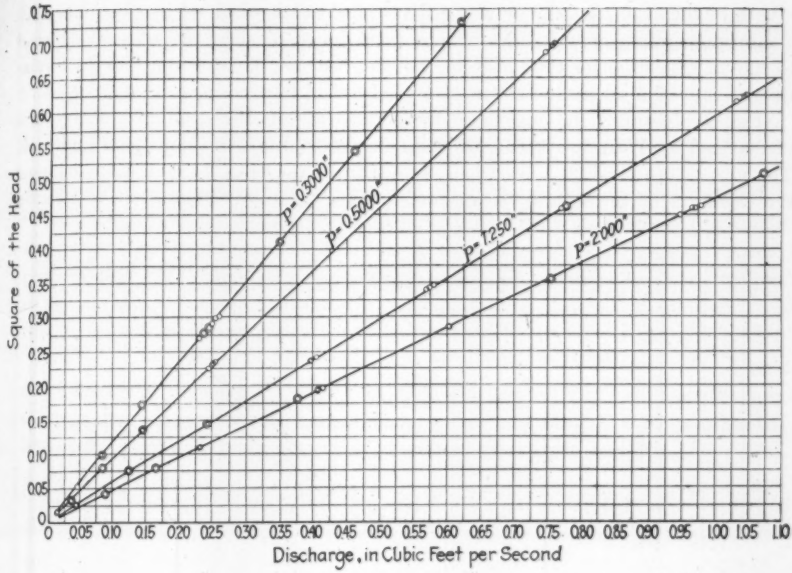
Square of the Head
Discharge, in Cubic Feet per Second
P= 0.3000'
P= 0.5000'
P= 1.250'
P= 2.000'

FIG. 7.

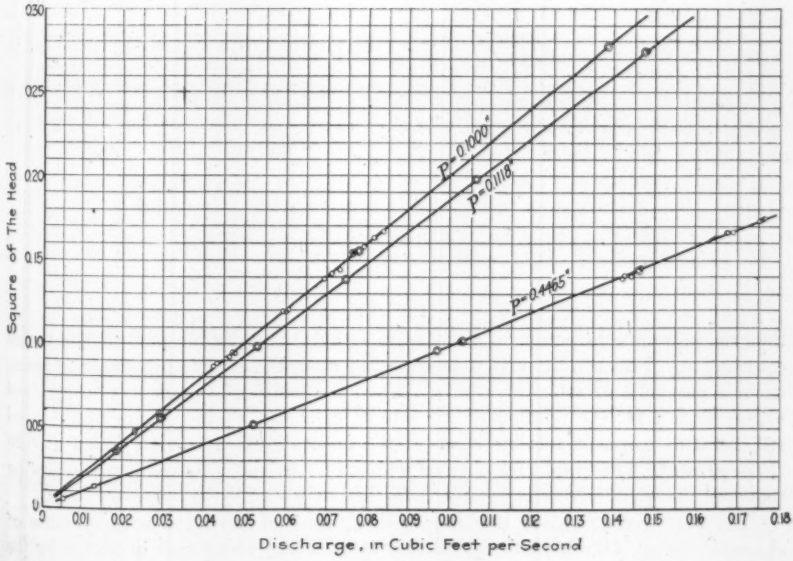
Square of The Head
Discharge, in Cubic Feet per Second
P= 0.1000'
P= 0.1118'
P= 0.4465'

FIG. 8.

Figs. 7 and 8 show that the rate of discharge bears a constant relation to the square of the head for any one weir with beveled notch when set in a channel with a minimum width of 4 ft. No graphs were drawn to show the relation of head to rate of discharge for the unbeveled notches because, as is proven later, the relation of rate of discharge to the square of the head is a variable in such cases.

Fig. 9 illustrates the change in the actual rate of discharge with the square of the head for Weir No. 0.4465, beveled notch, with different channels of approach. The graphs are straight lines when the width of the channel is 4 ft. Reducing the elevation of the weir crest appears to have no appreciable effect, indicating

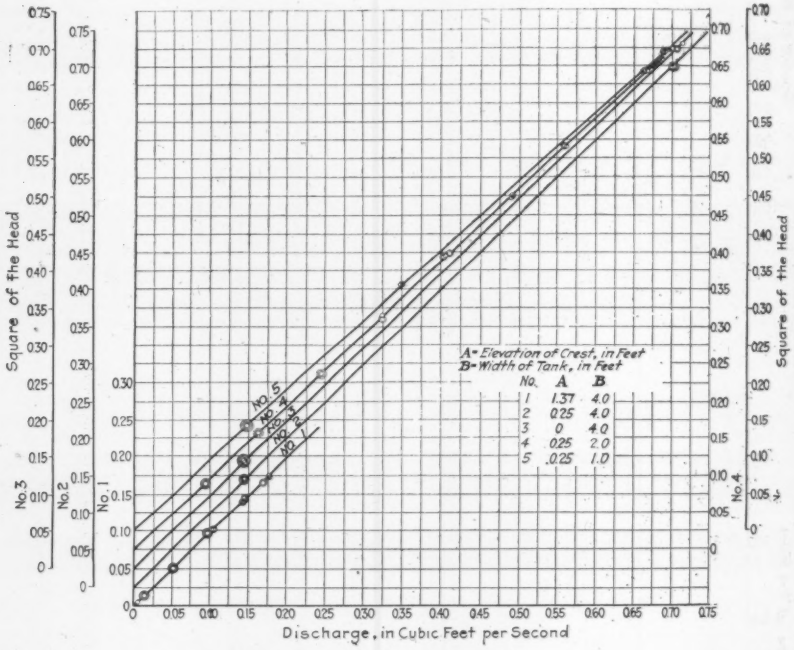

FIG. 9.

that the bottom contraction of the stream is only a negligible part of the entire contraction. This condition is to be expected when the crest is practically a point, as in a parabolic or triangular weir. When the width of the channel is reduced to less than 4 ft., the relation of the rate of discharge to the square of the head still remains constant for low heads if the crest elevation is maintained at 0.25 ft., or greater. The graphs for channel widths of 1 ft. and 2 ft. are curved, showing that the rate of discharge is greater than that indicated by the observed head.

The degree of curvature increases with increase of head and with decrease of channel width. This increase in the rate of discharge above that indicated by the elevation head is due partly to the velocity head in the channel and partly to the

change in stream contraction after passing through the weir. No attempt was made to distinguish between these two influences. Computations based upon the small amount of data available for the tests, with a constant crest elevation of 0.25 ft., seem to show that the total head is equal to $h + \frac{V_1^2}{2g}$, where h is the elevation head and $\frac{V_1^2}{2g}$ is the velocity head in the channel, both expressed in feet of water. However, the data are too meager to prove that the coefficient of the velocity head is unity. The gauges showed no drop in level of the water surface. It is the writer's opinion that channels of approach should be designed so as to preclude any appreciable velocity for two reasons: (1), the uncertainty of the effect of the velocity of approach; and (2), the inaccuracy in reading the gauges due to disturbance on the water surface.

The relations of the coefficients k and C to the head are illustrated in Figs. 10 to 12, inclusive, for all weirs other than Weir No. 0.4465. These coefficients are constant for any given weir with beveled edge; at least, for heads of 0.15 ft. and greater. This fact has already been established mathematically in the discussion of the relation of the rate of discharge to the square of the head. The coefficients are variable for Weirs Nos. 0.1000, 0.1118 and 2.000 with unbeveled notches, Figs. 10 and 11, increasing with decrease of head. The variation in the coefficients is gradual until the head is reduced to approximately 0.2 ft., when the curvature of the graph rapidly increases. The graphs appear to indicate a constant value of the coefficients at high heads although the experiments were insufficient to conclusively establish this point. There is less variation between the coefficients for beveled and unbeveled conditions with Weir No. 0.1000 than for No. 0.1118 or No. 2.000. This is explained by the fact that the thickness of the weir plate for No. 0.1000 is less than that for the other two. The values of the coefficients are in each case higher for the beveled than for the unbeveled notches when the head is greater than about 0.2 ft. Suppression of the bottom contraction of the nappe occurs at a higher head for the latter than for the former condition, the rate of flow being increased by suppression of the contraction. It will be noticed that the distribution of the plotted points is more uniform in the case of the beveled notches.

Considerable air was entrained in the water before experiments were begun on Weir No. 0.1118 with unbeveled notch. It was interesting to note at the time that small air bubbles, resembling miniature white beads, were arranged between the nappe and the down-stream edge of the notch.

Figs. 13 and 14 show the relation of the coefficients to the head for Weir No. 0.4465 for the several channels of approach. The graphs are straight lines for a channel width of 4 ft. and a minimum crest elevation of 0.25 ft. The coefficients are slightly higher when the crest elevation is zero for the same width of channel. This slight change in the coefficients, however, is insufficient to make itself felt in the relation of the rate of discharge to the square of the head. The coefficients are variable for the channels of 1-ft. and 2-ft. widths, as explained in the discussion of Fig. 9.

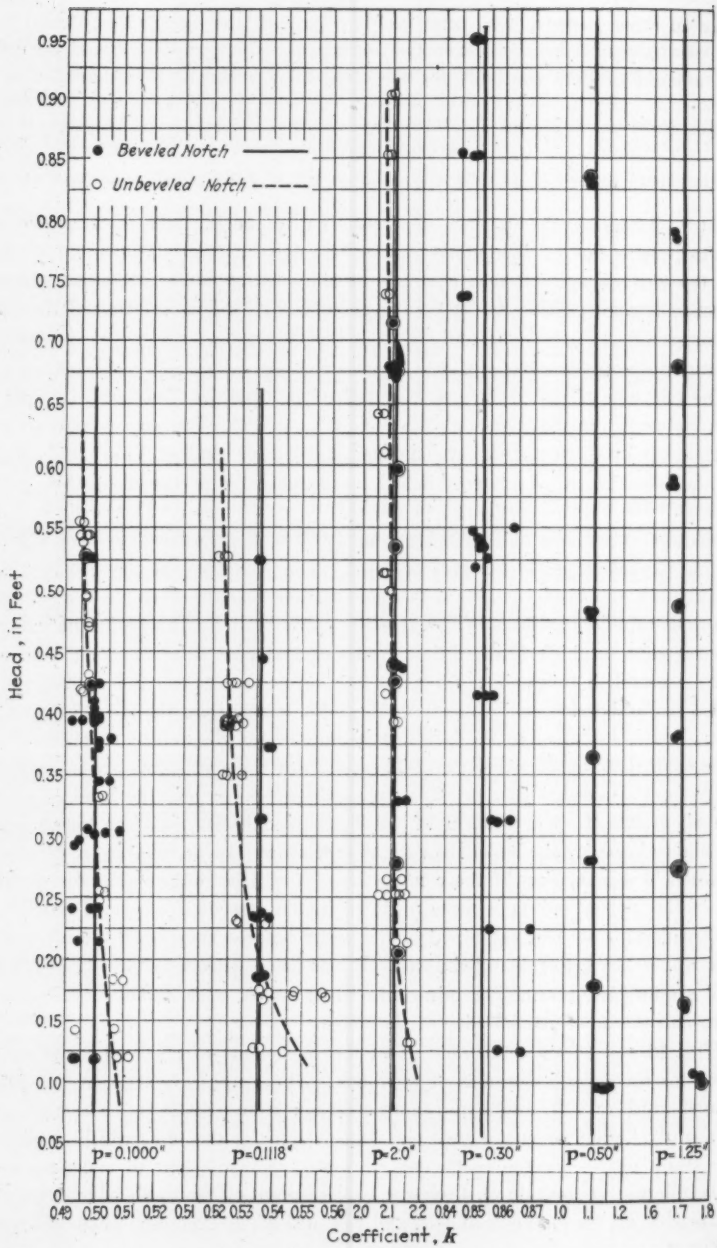
Beveled Notch
Unbeveled Notch
Head, in Feet
0.95
0.90
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0
p= 0.1000"
p=0.1118"
p=2.0"
p= 0.30"
p=0.50"
p= 1.25"
0.49 0.50 0.51 0.52 0.51 0.52 0.53 0.54 0.55 0.56 2.0 2.1 2.2 0.84 0.85 0.86 0.87 1.0 1.1 1.2 1.6 1.7 1.8
Coefficient, K

FIG. 10.

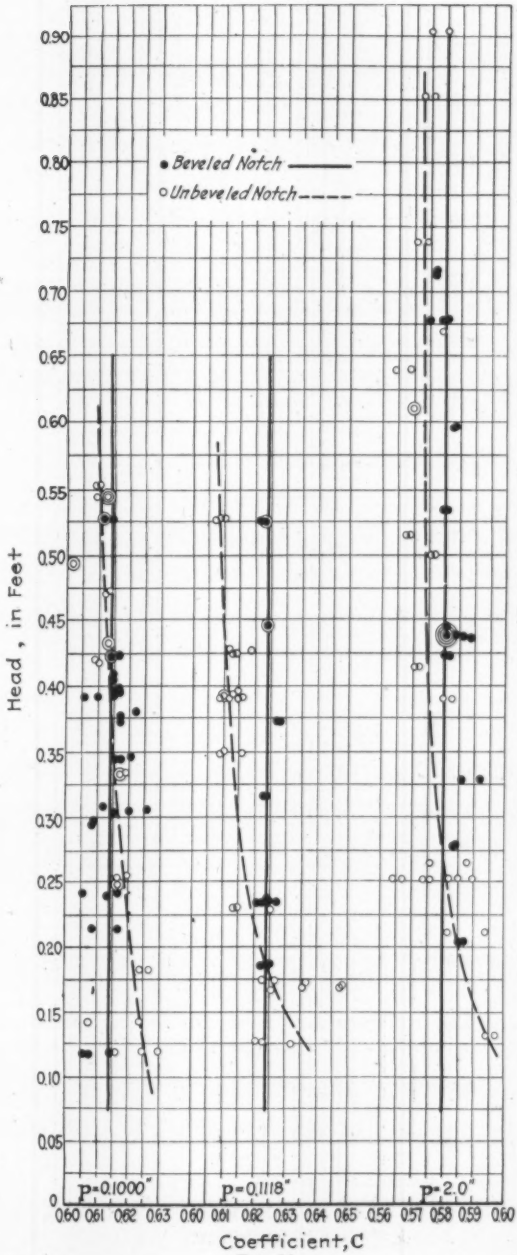
Beveled Notch
Unbeveled Notch
Head, in Feet
0.90
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0
P=0.1000"
P=0.1118"
P=2.0"
0.60 0.61 0.62 0.63 0.60 0.61 0.62 0.63 0.64 0.65 0.56 0.57 0.58 0.59 0.60
Coefficient, C

FIG. 11.

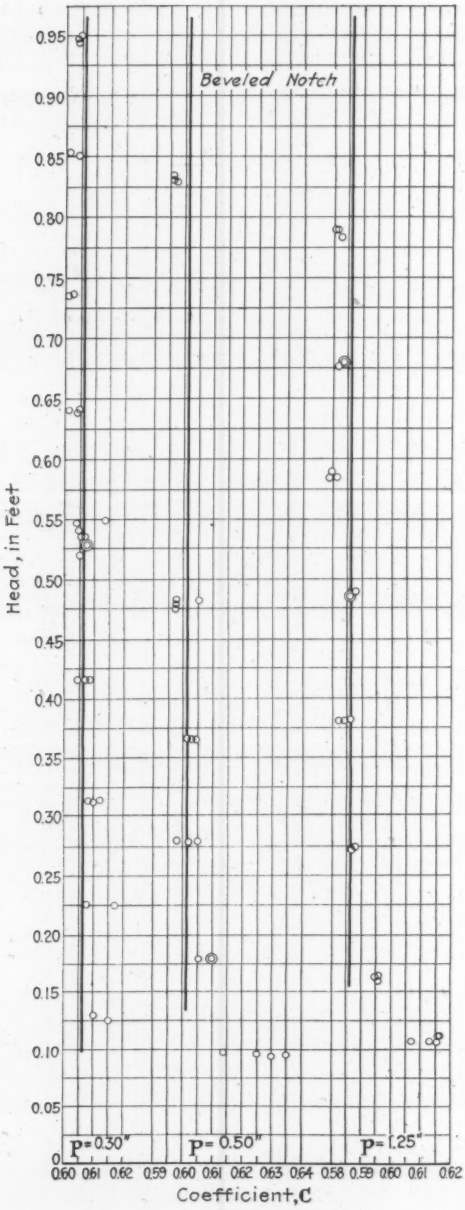
Beveled Notch
Head, in Feet
0.95
0.90
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0
P=0.30"
P=0.50"
P=1.25"
0.60 0.61 0.62 0.59 0.60 0.61 0.62 0.63 0.64 0.58 0.59 0.60 0.61 0.62
Coefficient, C

FIG. 12.

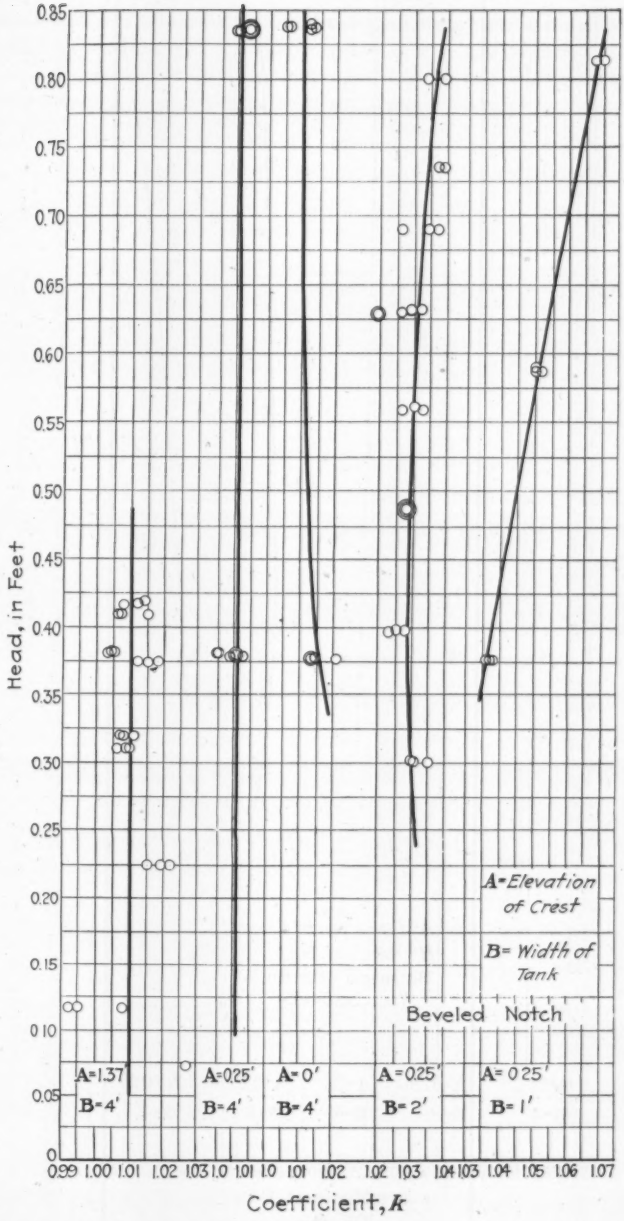
Head, in Feet
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0
0.99 1.00 1.01 1.02 1.03 1.0 1.01 1.0 1.01 1.02 1.02 1.03 1.04 1.03 1.04 1.05 1.06 1.07
Coefficient, K
A=Elevation of Crest
B=Width of Tank
Beveled Notch
A=1.37' B=4'
A=0.25' B=4'
A=0' B=4'
A=0.25' B=2'
A=0.25' B=1'

FIG. 13.

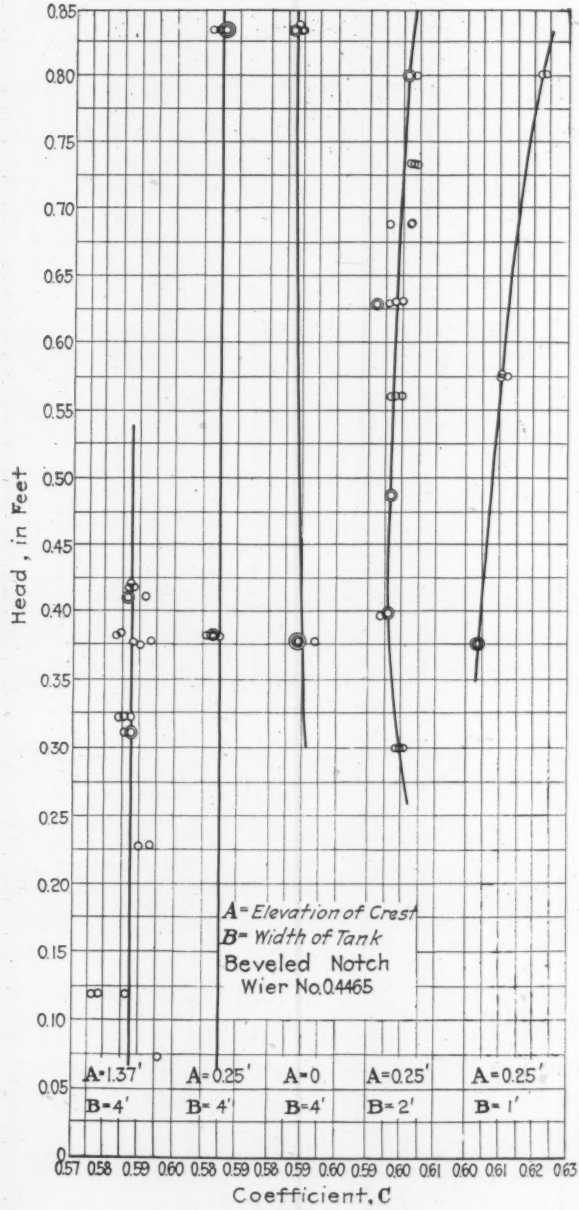
Head, in Feet
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0
0.57 0.58 0.59 0.60 0.58 0.59 0.58 0.59 0.60 0.59 0.60 0.61 0.60 0.61 0.62 0.63
Coefficient, C
A = Elevation of Crest
B = Width of Tank
Beveled Notch
Wier No. 0.4465
A=1.37' B=4'
A=0.25' B=4'
A=0 B=4'
A=0.25' B=2'
A=0.25' B=1'

FIG. 14.

One of the most valuable features of the parabolic weir is illustrated in Fig. 15, in which the variation of the square root of p with the coefficient k is plotted on logarithmic paper. The graph is a straight line, its equation being,

$$k = m\,(\sqrt{p})^n = 1.512\ p^{0.478}$$

This shows that the value of k can be readily predicted for any given value of p, and both theory and calibration tests indicate that the equation should hold true for all values of p.

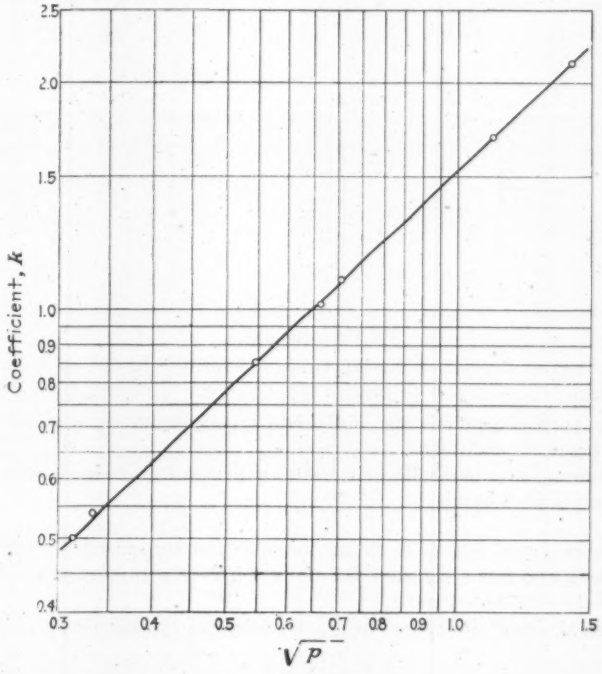

FIG. 15.

Table 3 shows the variation of the coefficients k and C with p for the seven weirs with beveled notches.

TABLE 3.—VARIATION OF k AND C WITH p.

No. p, in inches.	$\sqrt{p}$	k	C
0.1000	0.3162	0.500	0.615
0.1118	0.3344	0.533	0.619
0.3000	0.5477	0.852	0.605
0.4465	0.6682	1.01	0.588
0.5000	0.7071	1.10	0.605
1.250	1.118	1.69	0.588
2.000	1.414	2.11	0.585

Table 4 is a recapitulation of the relations of the coefficients to p as obtained from the original data.* Values of k were noted from the graph in Fig. 15 for values of p between the limits of 0.1 and 2.0 in. The values of C and K were computed from the equations

$$K = \frac{\pi}{4}\sqrt{2g}\,\sqrt{2p}\,; \text{ and } C = \frac{k}{K}.$$

TABLE 4.—RECAPITULATION OF k, K, AND C.

$$q = k\,h^2 = C\,\frac{\pi}{4}\,\sqrt{2g}\,\sqrt{2p}\,h^2 \quad Q = K\,h^2 = \frac{\pi}{4}\,\sqrt{2g}\,\sqrt{2p}\,h^2.$$

No.	p, in inches.	k	K	C
1	0.1	0.502	0.813	0.617
2	0.2	0.700	1.15	0.609
3	0.3	0.850	1.41	0.603
4	0.4	0.976	1.63	0.600
5	0.5	1.09	1.82	0.600
6	0.6	1.19	1.99	0.597
7	0.7	1.28	2.15	0.595
8	0.8	1.36	2.30	0.592
9	0.9	1.44	2.44	0.590
10	1.0	1.51	2.57	0.587
11	1.25	1.68	2.88	0.585
12	1.50	1.84	3.15	0.584
13	1.75	1.98	3.40	0.583
14	2.00	2.11	3.64	0.581

CONCLUSIONS.

The writer, as a result of the experiments described, has reached the following conclusions:

1.—The parabolic weir is one of the best types yet developed and is adaptable to a wide range of conditions.

2.—By means of a parabolic weir, the actual rate of discharge can be computed by the very simple formula:

$$q = k\,h^2$$

where q is the actual rate of discharge, in cubic feet per second, h is the elevation head, in feet, and k is a constant for any given weir with beveled notch. Values of k are given in Tables 3 and 4.

The actual rate of discharge can also be computed by the equation,

$$q = C\,\frac{\pi}{4}\,\sqrt{2g}\,\sqrt{2p}\,h^2$$

where C = the coefficient of discharge (Tables 3 and 4);
$\pi = 3.1416$;
$g = 32.16$ ft. per sec. per sec.;
p = variable in the equation, $x^2 = 2py$;
h = elevation head, in feet.

3.—The parabolic weir is adaptable to a simple and accurate device for recording the discharge.

4.—The values of the coefficients can be accurately predicted for any given value of p.

It should be emphasized that: (*a*) Beveled notches and thorough baffling are essential; (*b*) the velocity of approach should not exceed 0.5 ft. per sec.

* On file in the Engineering Societies Library, and to be published in full in a bulletin of the Engineering Experiment Station, Purdue University, Lafayette, Ind.

APPENDIX.

Sample Computations.

Weir No. 0.1000; beveled notch.
Run No. 1.
Width of channel = 4 ft.
Elevation of the weir crest above the bottom of the channel = 1.37 ft.
Observed head, $h = 0.527$ ft. = gauge reading minus zero head.
Time = $t = 232.2$ sec.
Total discharge = 2 000 lb.

$$q = \frac{\text{Lb.}}{62.3\ t} = \frac{2\ 000}{62.3 \times 232.2} = 0.138 \text{ cu. ft. per sec.}$$

The weight of a cubic foot of water has been assumed as being equal to 62.3 lb. for slide-rule accuracy.

$$k = \frac{q}{h^2} = \frac{0.138}{0.2777} = 0.497$$

$$C = \frac{q}{Q} = \frac{k\,h^2}{\frac{\pi}{4}\sqrt{2\,g}\sqrt{2\,p}\,h^2} = \frac{k}{K} = \frac{0.497}{0.813} = 0.611$$

The three center of gravity points for all the runs are listed in the following tabulation:

q	h^2	k
0.0357	0.0713	0.500
0.0628	0.1255	0.500
0.0898	0.1798	0.500

k is a constant, demonstrating that the graph is a straight line, for its slope is everywhere the same.

DISCUSSION

B. F. GROAT,* M. AM. SOC. C. E. (by letter).—Mr. Greve's paper is a valuable contribution to the subject of hydrometrics. The writer knows well the difficulty of conducting a long series of tests and maintaining the necessary precision in all departments. Hence, there is always room for pertinent criticism even in the case of the most painstaking care. The comments to be made, however, apply for the most part to the great majority of past observations, and are offered only with the view to emphasis on the need for more data in future hydrometrical experiments.

Mr. Greve uses a theory which has been applied in deducing the formulas for weirs and orifices for many years. As is well understood, this theory omits all questions of viscosity, temperature, surface tension, conditions of the channel of approach, and character of the notch. It also involves the curvature of the surface of the water, as it nears the weir, implicitly. Hence, for example, it is not safe to conclude that k is constant for any range of heads on the same parabolic weir, or that it is connected with a root of p by a straight-line relation because it shows these properties in a formula deduced by means of a partly incomplete theory. For example, in Figs. 10, 11, and 12, it seems clear that k and C decrease with increasing head. At least in the case of Weirs 2.0, 0.3, 0.5, and 1.25, it is apparent for heads above the limit of 0.15 ft. set by Mr. Greve. This applies in the beveled as well as in the unbeveled condition of the notch. As regards the condition of the edges of the weirs, it would be helpful, as well as more explicit, if Mr. Greve would state exactly what is meant by "the purpose of the experiments with the unbeveled openings was to determine the necessity of a sharp-edged weir. The bevel was made at an angle of 30° to the plane of the weir, allowing a thickness of metal of $\frac{1}{32}$ in. in contact with the water." As Mr. Greve concludes on page 508 that "beveled notches and thorough baffling are essential", it is perhaps proper to suggest that he illustrate just what the condition of the notch was, both in the beveled and unbeveled states. A very simple sketch would explain it perfectly.

It was at first hoped that the paper would throw more light on the question of hydraulic similarity.† Although it is true that all parabolas are similar, one to another, there are so many other details of weirs which need attention when attempting to show similarity by experiment, that it is scarcely possible to find tests for other purposes lending themselves to this object. Perhaps it may be worth while to suggest that in the future other experimenters give attention to the possibilities of their work in the broadest sense by collecting all such data which may be economically accumulated. In the case of weirs, for example, it has been too common in the past for the observers not to record the temperature of the water in the various experiments. It is a great mistake to neglect this, for the discharge is considerably affected by temperature under many circumstances, and past records, such as there are, are discordant.‡ J. G. Mair's experiments (quoted on page 40 of the article just referred to) seem to show that the coefficient of discharge for circular orifices has a minimum, just as Bazin's weir tests showed a minimum for the weir coefficient

* Pittsburgh, Pa.

† See *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1138, and *Canadian Engineer*, November 25th, 1920.

‡ See the article on Hydraulics, *Encyclopædia Britannica*, 11th edition, Vol. 14, p. 42.

(page 50). The head, size of notch, and conditions of edge are separate variables and the tests should separate their effects as far as possible.

Theoretically, for given viscosity, the coefficient of discharge should increase with increasing size of notch. This may be inferred from Equation (19) of the writer's paper in *Transactions* to which reference has been made, for a fixed coefficient requires that the smaller opening should attach to the lesser viscosity, and it is known that the coefficient increases for a given opening when the viscosity decreases. Furthermore, since the viscosity of water decreases with increasing temperature, it may be seen that one can reasonably infer the separate effects of temperature, viscosity, and size of opening. Thus, such effects as those resulting from the condition of the edge and surface tension may account for the opposing tendency which produces the minimum as a net result of the action of all combined.

Bazin's minimum coefficient occurs at about 0.8 ft. for the standard weir. Since Mr. Greve's tests do not seem to have exceeded this head very much, it may be that the minimum values, if such really do exist, are more or less beyond the range of the experiments, possibly near the upper limit of head in the tests. As suggested for the case of the circular orifices just mentioned, it may possibly be that the reciprocal or inverse relation between p and C is due largely to the conditions of the notch, its edge, and other variables than p.

The most important, and at the same time the most difficult, achievement in tests on weirs and notches is the balancing of the precision of the observation of the values of the various quantities which enter the subsequent calculations. Where the discharge depends on the square of the head, it is clear that an error in a reading of head will be doubled, relatively, in the calculated square. Therefore, it will be doubled in the calculated discharge. When the head is only a fraction of a foot there ought to be some means for balancing the precision of the corresponding readings with that of the discharge, which latter, it would seem, would possess a far superior precision when such large quantities as 2 000 lb. were observed. There are three classes of errors to guard against: accidental errors are the least important and can be reduced by multiplying readings; constant errors of a test are troublesome and cause erratic plottings; constant errors of the apparatus are by far the most serious, and may require profound changes before they can be reduced. Constant errors of method are included in the last class of errors. If discharges are to be checked by the chemical method it must be remembered that changes in viscosity produce changes in discharge coefficients.

Mr. Greve has rendered a great service in publishing the unique results of his tests on parabolic weirs and it will be interesting to await similar experiments on such weirs of large size if it is found that they lend themselves conveniently to large discharges.

E. G. WALKER,* M. AM. SOC. C. E. (by letter).—This paper records a new departure in the form used for the profile of a gauging notch, the principal noticeable feature being that the notch is given a curved outline. In this respect it differs from those which have been used generally up to the present for practical work.

* London, England.

There is no doubt that the advantages claimed for the new notch are substantial. It is of considerable benefit to have a simplified formula for calculating the rate of discharge, which involves no greater complication than raising the head to the second power. The uniformity of the coefficient of discharge over considerable ranges is also a solid advantage, and the graphs shown in the paper indicate that this type of notch may be used to predict results with a fair degree of accuracy, particularly when the best proportions of height to width are chosen. As was no doubt expected, the sharpness of the edge of the notch has a very great influence on the flow and, in this respect, the parabolic notch does not appear to be appreciably different from the straight notch.

The principal disadvantage of this type of notch appears to lie in its relatively complex shape. The notches that have been used in practice hitherto are nearly always straight-edged, of rectangular, square, trapezoidal, or triangular outline, and it is obviously much easier to reproduce accurately a notch of one of the older types than one which has to be formed to a curved profile even though, as in the present case, this curve is a very simple one to set out. There is also the possibility that the vertex of the notch may not be so accurately defined in practice as in the case of the older types, particularly as the curvature of the parabola is changing rapidly at that point, and it would seem that errors in shaping the notch are more likely to occur here than at other parts of the outline. It is probable that a given amount of error would have a greater effect at this point than anywhere else. For this reason the very narrow notches with which the author has experimented do not seem to be so well adapted for practical use as those in which the ratio of breadth to height is greater.

R. L. SACKETT,* M. AM. SOC. C. E. (by letter).—This paper represents a valuable addition to the literature on devices for the measurement of the flow of water. Although the rectangular weir is satisfactory for large volumes where the flow is relatively constant and the weir is proportioned to such volumes, it is defective in that the coefficient varies as the hydraulic radius between wide limits. The V-notch has the advantage of a reduced variation in the coefficient and lends itself to automatic registering devices which show the rate of flow and the integrated flow. The parabolic weir, from the data contained in the paper, seems to be particularly well adapted to the use of such recording devices, because with the beveled weir the coefficients are practically constant for wide variations in head. Since the discharge varies as the square of the head, the logarithm of the discharge varies as twice the logarithm of the head, and, therefore, logarithmic paper can be used to advantage with recording apparatus. With the proper proportioning of lever arms the rate of discharge could be read directly and would be recorded directly on the paper. For accurate measurements no corrections would be necessary, except, perhaps, for temperature in case of handling hot water.

A weir of such form that the quantity of water discharged varied directly as the head would offer a slight advantage in that the area of the diagram which recorded the rate of discharge could be measured with a planimeter. The equation of such "a proportional flow weir" was first developed, as far as the writer knows, by Professor Rettger, of Cornell University.† However, the curve of the edges of

* State College, Pa.

† *Engineering News*, June 25th, 1914, p. 1409.

the weir is an asymptote to the crest and an adjustment is necessary, such as making a rectangular weir of very shallow depth with the "proportional" curve above. Tests were made at Cornell University of two such weirs.* Unfortunately, the coefficient of discharge varies from about 0.605 to nearly 0.700 in a variation of head of 1 ft. The author mentions a method of modifying the form so that the coefficient would be constant and the same process could be applied to the triangular weir, but the commercial uses to which the latter is put do not demand such refinements.

Early in 1916, Mr. H. C. Alger, of Chicago, Ill., devised a weir in which the discharge is theoretically proportional to the head. It is easy to show that if two cycloids are placed parallel, with a horizontal crest joining them, the discharge through the space between the cycloids will be proportional to the depth on the crest. Such weirs were tested, and the corrected slot was finally cut with jigs so that the coefficient was practically constant. This device lends itself to apparatus for recording the rate of flow and also is easy to integrate for total volumes.

In a paper entitled "An Improved Weir for Gauging in Open Channels",† Clemens Herschel, Past-President, Am. Soc. C. E., describes a weir which has a 2 to 1 slope of the bottom approaching the weir, a rounded crest, and a sloping apron below the crest. The discharge of this weir is stated as being proportional to the depth measured at the crest, but the information does not indicate clearly the variation in the coefficient for wide variations in depth. The nearest approach to such a weir of which we have information is given in the series of experiments by Bazin. In this case the coefficient varies from 2.75 to 3.73 with a variation in head from 0.3 to 1.3 ft.; for another, the coefficient varies from 3.14 to 3.66 with a variation in head of from 0.5 to 4.0 ft. For another, with a rounded crest and 1 to 1 slope of the bottom approaching the weir, the coefficient varies from 2.95 to 3.61 with a variation in head of from 0.5 to 4.0 ft. The question naturally arises, therefore, is the coefficient in the Herschel weir practically constant for ordinary test purposes in determining the flow through hydraulic turbines, the quantity of cooling water used in condensers, the condensation, or the return water from heating systems? Further information is needed.

So far as is known, the parabolic weir has two features which mark it as superior for the accurate measurement of wide variations in flow, namely, a relatively deep section at small discharges and a relatively large capacity for a given depth, that is, the area is small for low heads and increases faster than the depth. The coefficient is practically constant in the weir with a beveled edge, and the application of recording and registering apparatus is simple.

JOHN H. GREGORY,‡ M. AM. SOC. C. E. (by letter).—It is especially interesting to the writer to note that some one has at last made some actual experiments on parabolic weirs. The writer has long wished to make a set of experiments on this type of weir, but when he has had the time, the opportunity was lacking, and *vice versa*. There is much to be said for the parabolic weir, as evidenced by the experiments made by the author.

In the synopsis of the paper, the author states that the parabolic weir is the

* *Engineering News*, November 25th, 1915, p. 1018.
† *Journal*, Am. Soc. Mech. Engrs., February, 1921.
‡ Baltimore, Md.

invention of Mr. C. Lauritson, of the Ohio Body and Blower Company, Cleveland, Ohio. If by the term invention is meant the conception of the idea of a parabolic weir, it would be interesting to know the date, or approximate date, when that type of weir first occurred to Mr. Lauritson, and the writer would ask that said date be given by the author in his closing discussion.

The idea of the parabolic form of weir occurred to the writer more than twenty-five years ago, and he should not in the least be surprised if a similar idea had occurred to somebody else long before that. In a more or less varied correspondence on hydraulics and hydraulic problems with numerous engineers and investigators, covering quite a period of time, the writer has learned that thoughts and ideas which seemed new to him were not new, but had occurred to others previously. He has in mind one particular hydraulic diagram in which the method of plotting was entirely original with him, and yet he learned from one of the best known teachers of hydraulics and mechanics in the United States that a similar method of plotting had been used by another engineer at an earlier date. True, the units were different, but the fundamental method of plotting was exactly the same.

It was while the writer was an Assistant in the Designing Office of the Metropolitan Water Board of Massachusetts, on the staff of the late Frederic P. Stearns, Past-President, Am. Soc. C. E., then Chief Engineer of the Board, that the parabolic weir first occurred to him. Certain questions had arisen relative to mill damages due to the diversion of water, where there were two or three mill-ponds in series. Water was stored in these ponds during the night and drawn during the day. The data were as follows: Given a mill-pond of known dimensions, with flash-boards of a given length and depth and with water just level with the top of the flash-boards; there being no blow-off gates, if all the flash-boards of, say, the middle pond were raised at a given hour, how long would it take for the water level to drop to the crest of the flash-board weir, assuming, of course, no inflow?

It was while working out special weir problems of this kind that the thought occurred to the writer that a type of weir in which the discharge varied as the square of the head might be of value. Accordingly, in his spare time, he worked out the fundamental formula for the discharge over a parabolic weir, and with the necessary change in units his formula corresponds exactly with Equation (16) given by the author. At the time, the mathematical work was done either in some computation book or on a loose sheet, but, at a later date, this was copied into a bound notebook, and by reference to this notebook the writer finds the following dates at the bottom of the page: "Mar. 21, 1898. Demonstrated Dec. 3, 1895".

Since that date, the parabolic weir has recurred to the writer many times. About ten or fifteen years ago, he suggested to one of the members of the faculty of the Massachusetts Institute of Technology, either Professor Dwight Porter, M. Am. Soc. C. E., or one of his associates, that a series of experiments on parabolic weirs would make an interesting subject for a thesis; so far as the writer knows, however, no experiments were made.

In more recent years, the writer again suggested a series of such experiments, this time, in 1918, to the late Professor Grandville R. Jones, Assoc. M. Am.

Soc. C. E., of the Johns Hopkins University, in response to an inquiry by Professor Jones for suggestions as to experiments which might be carried out in the hydraulic laboratory then under his direction. The matter was discussed with him in more or less detail, and only for Professor Jones' untimely death the writer feels sure that the programme roughly outlined would have been carried out.

It is exceedingly doubtful whether either Mr. Lauritson or the author ever heard, even indirectly, of any of the facts mentioned, and they have been set forth merely as a matter of record. The writer does not wish to detract in the least from the great credit due both Mr. Lauritson and the author for their valuable contribution to hydraulic literature; he only regrets that he did not have the opportunity of co-operating with them.

F. W. GREVE,* Esq. (by letter).—The writer acknowledges with thanks the helpful suggestions and criticisms contributed in the discussion of his paper.

Mr. Groat is quite right in his criticisms that the theory commonly used in deducing the formulas for weirs is unsatisfactory. The writer made use of this method of deduction only to determine the probable shape of a notch in which the discharge varies as the square of the head. The conclusions drawn, especially as regards the constancy of the coefficient, k, for a given parabolic weir and its connection with a root of p by a straight-line variation, are not based on theory, but on the results of the experiments. In short, the relations established are entirely empirical.

Parabolic weirs with unbeveled notches can be constructed at a considerable saving over those of the beveled type. Experiments on the former type were made to determine whether beveling was essential to meet the requirements set forth under the purposes of the investigation (page 488), and to ascertain the variation in the coefficient of discharge with the thickness of the edge in contact with the issuing stream. The results demonstrated that the beveled notch was not satisfactory. Fig. 16 illustrates the notch both beveled and unbeveled.

HORIZONTAL SECTION, EDGE OF NOTCH

FIG. 16.

Replying to Mr. Gregory's inquiry, the writer will state that the term "invention" is used to denote the fact that Mr. Lauritson was the first to apply for, and receive, a patent on the parabolic weir. Several years ago the calibration of parabolic weirs was listed as a thesis subject in the Hydraulic Laboratory of Purdue University, but, unfortunately, the experiments were not undertaken. Undoubtedly, the conception of this particular type of notch has occurred to others besides Mr. Lauritson, Mr. Gregory and the writer.

* Lafayette, Ind.

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MUNICIPAL ENGINEERING

ADDRESS AT THE ANNUAL CONVENTION AT HOUSTON, TEXAS, APRIL 27TH, 1921

BY GEORGE S. WEBSTER,* PRESIDENT, AM. SOC. C. E.

In the performance of my duty as President to deliver an address at the Annual Convention I have selected the subject "Municipal Engineering", in which work I have been engaged during all my professional career. While my remarks may in some measure reflect conditions as I have observed them during my connection with municipal management in the City of Philadelphia for more than four decades, I believe these conditions are largely typical of those in other American cities.

The term "Municipal Engineering" did not come into general use until the latter part of the Nineteenth Century; it is that branch of Civil Engineering especially related to the problems of municipal corporations, and includes the planning, construction, and operation of public improvements and utilities required for city growth and development, for furnishing the citizens and industries with certain commodities needed for health, commerce, and prosperity, and for removing and disposing of the wastes which are detrimental to health and to the well being of the people.

ORIGIN OF MUNICIPAL ENGINEERING.

The rapid increase in urban population, with the consequent problems involved, has necessitated making adequate provision for the public service required for the social, business, and industrial enterprises of these people and for their comfort, convenience, and safety, and has been the chief agency in developing Municipal Engineering and raising it to a position of importance equal to that of other branches of the Profession.

In the days of the small city, as transportation, sanitation, water supplies, and other public works first became essential, great waste often existed in public improvements by reason of the fact that there was an absence of a comprehensive understanding of the problems and of their proper solution. Important public

* Philadelphia, Pa.

works were often in charge of men selected without regard to their training and qualifications for the work. Hence, the development and operation of cities were not always conducted efficiently.

As cities grew, and the demand for improved streets, sanitary drainage, water supplies, and other public service increased, greater engineering skill was required to meet these demands, until at this time, cities have become such tremendous working organisms, and their problems of physical development so varied and complex that the chief engineer of the modern city, to meet his responsibilities, must have skill in a wide field of engineering. He must also have foresight and initiative in discerning and anticipating the evolution of engineering enterprises as they affect the progress and prosperity of the city. Although most of the work connected with the growth and development of cities comes under the province of Civil Engineering, yet the increasing importance of specialization has developed the distinctive branch of Municipal Engineering.

South Philadelphia Improvement.

In Philadelphia, just prior to the breaking out of the World War, there was organized, officially authorized, and financed, the most comprehensive city-planning project ever actually undertaken as a single co-ordinated enterprise in the United States. This is known as the South Philadelphia Improvement, and affected an area of more than 8 sq. miles. It provided (1) for a revision and extension of the street system in accordance with the modern theory and practice of city planning; (2) for such a change in the methods of land subdivision as would insure better homes for the people than it had been customary to build in that section of the city; (3) for a liberal apportionment and systematic distribution of parks, parkways, playgrounds, public squares, and other open spaces, and for local, civic or neighborhood centers; (4) for industrial and commercial areas; (5) for a complete drainage system of an unusually complex nature; (6) for the complete relocation and reorganization of a network of main and branch railroad lines and freight yards belonging to two great railroad systems and their reconstruction under a joint traffic agreement permitting their use by foreign companies; (7) for the abolishment of grade crossings, the construction of elevated railroads and railroad and highway bridges; (8) for the construction of terminal yards and piers for the transfer of rail and ocean freights; (9) for harbor improvements; (10) for the development of wide marginal ways along the harbor for commercial uses; and (11) for the acquisition of large areas of harbor frontage and the construction of an extensive system of municipally owned and operated piers.

The work authorized and commenced under ordinances, contracts, and agreements of the city and the railroad companies, called for an expenditure of more than $25 000 000, all of which was pledged and available. Had the war not intervened, this work, which was little more than the beginning of a great project of comprehensive city development, would now be completed.

All the preliminary investigations and studies and all the plans, estimates, contracts, ordinances, and agreements involved in the conception, promotion, and execution of this many-sided project, were prepared by the Engineering Staff of the city, except such as related wholly to the construction of railroad lines, yards,

and piers, although when such construction affected the public or city's interests it was subject to inspection and approval by the municipal engineers.

The completion of this project has been deferred but not abandoned and will undoubtedly go forward as soon as the railroad companies are again on a safe financial footing. It is referred to here as being perhaps the best practical illustration that can be given of the scope and range of Municipal Engineering in a great city and of the obligations and responsibilities of the Municipal Engineer.

Growth and Development of Municipal Engineering.

To consider properly the growth and development of Municipal Engineering it is necessary to consider them under the following divisions.

First.—The functions which have to do with the development and growth of cities:

a.—City planning.
b.—Transportation.
c.—Port development.
d.—Bridges.
e.—Street pavements.

Second.—The means for bringing to the citizens the publicly furnished commodities:

f.—Water supply.
g.—Gas and electricity.

Third.—The means for removing wastes from the city:

h.—Sewerage.
i.—Refuse disposal.

City Planning.

During the past quarter of a century the evolution of cities and the revelation of the disastrous results that have sprung from the lack of intelligent control and co-ordination of the elements of urban growth have brought forward a new constructive activity, namely, city planning. Although this new activity calls to its service the skill and genius of all the professions, arts, and sciences which can contribute to city building, its ultimate success, like the ultimate success of any great forward step in constructive progress, depends on the co-operation of the engineer, and particularly of the Municipal Engineer, with his knowledge of the present and future needs of the city. Through his daily contact with its problems, he knows what standards of public works and service are necessary for the welfare of the people and the best expression of community life.

Fundamentally, city planning has to do with the laying of the foundation of the city, on the stability of which the usefulness of the structure raised depends. The failure of the city structure based on a small-town foundation to carry the service load of the modern city has been the real driving force behind the city-planning movement of the past twenty-five years.

The street system is the basis of every city plan and is the controlling element in determining the efficiency and economy of public works; its great importance lies in the fact that it provides channels for traffic, facilitates the subdivision and use of land, and creates opportunities for the expression of civic art. It is

the most difficult and costly part of the city's structure to change after it has been fully improved.

Most of the cities of America began their existence with an orderly layout of streets over an area of such extent as the founder or founders considered necessary for the enterprise they had in mind; when the growth reached the limits of this orderly layout it was almost invariably extended into new areas without any regard for order, system, or topography. Cities have usually grown by building up land subdivision after land subdivision on private initiative, each independent of all the others, with streets considered only in their effect on the salability of lots and without regard for their traffic value.

Until within a comparatively recent period it was customary in the initial layout of a town site to establish a severely rectangular system of streets regardless of topography and to adopt arbitrary and uniform standards for street widths and block dimensions. This custom has imposed on cities which have grown to any considerable size, very complex and costly problems of replanning and reconstruction which the Municipal Engineer is called on to solve. Transportation and drainage—both engineering problems of the greatest importance in economic city development—were virtually overlooked, to become later the subjects of controversy and large expenditures. Much of this could have been avoided by the exercise of engineering skill and foresight in laying out the streets in such a manner as to permit of transportation lines and drainage channels being constructed along direct lines of least resistance.

An economic street system calls for such an arrangement of its units and such a differentiation of their widths as will reduce to a minimum the amount of land used for such purposes and still provide sufficient street areas in proper locations to care adequately for all the services of every character which may be placed in or on or above them. This calls for a most thorough and forward-looking survey by the Municipal Engineer of the probable nature and extent of the city's growth, and the character and amount of service required of the streets to meet the needs of that growth. The street system, or at least those streets which are to serve as main thoroughfares or arteries of city circulation, should be planned for in advance of other urban improvements and in a manner to meet all future requirements.

City Transportation.

The traffic on the highways of our cities has grown so rapidly in the past few years, especially since the advent of the automobile and motor truck, as to become a serious problem, claiming the earnest attention of the authorities. As our cities continue to grow, and as trade and business continues to be increasingly concentrated at one or more centers, the problem of transportation of all kinds through the city becomes more difficult. The economical administration of industry and the transaction of business in cities are dependent on the ease with which materials and merchandise may be moved, for modern business requires that transportation shall be by the most direct route and in the shortest time.

Inseparably interwoven into the street system and its functions are the facilities of transportation, whether they relate to the local movement within the city itself or to that wider movement through which the city maintains its contact with the world. Transportation is essentially a problem of planning

constructive and administrative engineering with which the Municipal Engineer must keep in touch if his plans for city development are to function smoothly. Rapid and convenient urban and interurban service depends on the extent to which the system of main traffic streets has been planned to connect the important city centers by the most direct and adequate routes. The vigilance and persistence of the Municipal Engineer is necessary in securing the proper adjustments of both street and railroad grades and in preventing or avoiding the cutting off or abandonment of important streets in the construction of railroads and yards. In devising plans for getting rid of the grade crossings which are still a menace to life and property and an obstruction to traffic in many of our cities, he is the most important representative of the public.

Port Development.

Nearly all the large cities of this country are located on navigable waterways—many of them being situate on deep estuaries leading direct to the ocean. The World War has resulted in the creation of a great international trade between this and foreign countries. To maintain this trade successfully in competition with other countries, it is necessary that the most modern facilities for handling and shipping goods shall be provided. Port authorities in every city on the Atlantic, Pacific, and Gulf Coasts, and on our Great Lakes, anxious to share in this foreign trade, have been actively engaged during the past few years in developing their terminal facilities and are now planning greater extensions to handle the additional water-borne cargoes. In order that a port may compete in the world trade it is essential that provision shall be made in the planning of the city for the great trunk railroads to reach the water-front, either directly or over a belt line railroad system, so that the cars may deliver cargo at the ship's side. It is also necessary that a system of traffic streets shall be laid out and developed in the rear of the piers and along the water-front, to give highway facilities for motor trucks and vehicles to make deliveries to and from the industries, warehouses, and stores located in the vicinity of the water terminal.

The authority to plan and administer the ports of this country is vested generally in the officials of the city, although there are several instances where ports are under the control of State commissions; but in all cases the development of the land side of the port is a proper task for men skilled in both city planning and other municipal work.

City Bridges.

No construction work attracts more attention and receives more comment from the public than the bridges which the city engineer designs and constructs. In no other branch of the Profession has there been greater progress, especially along esthetic lines, than in the art of bridge building. It was formerly the usual practice when an important bridge was to be erected to specify the needs in the way of travel and loading and to ask contractors and bridge-building companies to submit bids with plans and specifications for the type of structure they proposed to erect for the prices bid. This method did not always prove satisfactory to either party, and was found to be uneconomical. Now, in most of the large cities, the engineering forces of the municipality prepare plans for the bridge to be erected, giving every detail of construction, with specifications defining the

quality of the materials to be incorporated in the work, as well as the character of the workmanship; this results in real competition and economy in construction. The same department supervises the construction and is generally charged with the maintenance.

The utilization of concrete and reinforced concrete in recent years marks an epoch in the art of bridge building and has resulted in great improvement in the architectural features and appearance of bridges. Artistic cornices, projections, balustrades, and other ornamentation can be obtained with concrete at little expense. This fact makes concrete a valuable and desirable material for use in building bridges in public parks, in suburban and residential sections, and along parkways and boulevards. Engineers were formerly content to build bridges for their strict utilitarian use, but at present, particularly since the advent of concrete, greater attention is being given to the architectural features.

Street Pavements.

Fifty years ago the paving and maintenance of the highways in many cities frequently were in charge of men unskilled, and selected by political preferment. Now, due largely to the activities of civic and business organizations interested in street betterments, and also to the advancement in municipal administration, work of this kind is usually entrusted to trained engineers familiar with municipal affairs.

The development and increase of motor traffic in cities has led to an improvement in its pavements to meet this demand. Materials heretofore found to be satisfactory have proven to be inadequate, and this has necessitated the development of road surfaces which will give maximum wear with a minimum cost of construction and maintenance. Laboratories have been established for research and to provide means for determining the properties of materials. Comprehensive specifications are now drawn in which the materials to be used are definitely described and the methods of tests to insure such materials are clearly set forth. This enables the city to obtain proper construction of its street surfaces and to effect great economies, due to the fuller and freer competition of bidders, and greater permanency of the work.

Water Supply.

Providing purified water supplies and comprehensive sewerage systems are the principal functions of Municipal Engineering which have a direct bearing on public health. As recently as the middle of the Nineteenth Century, these two great questions were not always considered as engineering problems.

Private supplies of water were obtained ofttimes from dug wells in close proximity to the dwelling-house; excreta were disposed of in privy vaults and cesspools which, through underground or surface overflow, made possible the contamination of the near-by water well.

It had long been the custom to construct rough masonry culverts to enclose the streams flowing through the town, and it gradually became the practice to permit overflows from cesspools and also connections from water-closets to be made to these crude storm-water drains, thus conveying the sewage to a water-course from which a public water supply was taken without even a thought

of purification. As the cause of typhoid fever and other water-borne diseases was not known at that time, little heed was paid to such contamination.

Water Filtration.

Filters for the improvement of public water supplies were first used in Europe, and, about 1866, the late J. P. Kirkwood, Past-President, Am. Soc. C. E., went abroad for the purpose of examining such works. Based on the data he obtained, municipal water filters were constructed and successfully operated at Poughkeepsie, N. Y. The Massachusetts State Board of Health, in 1890, began experiments on the filtration of water, and shortly afterward municipal water filters were designed for Lawrence, Mass., based on these experiments. The filtration of the Merrimac River water at Lawrence demonstrated the beneficial effects on the public health of purifying water by filtration, not only as evidenced by the reduction in typhoid fever, but also by the lowering of the total death rate.

When the City of Louisville, Ky., conducted experiments in 1895 on the filtration of the Ohio River water, it was found that the practice of sedimentation and slow sand filtration known at that time was not applicable to a raw river water containing large quantities of very finely divided suspended matter. In these experiments there was developed the chemical coagulation of the raw water and its subsequent mechanical or rapid filtration now so commonly used.

About 1908 the sterilization of public water supplies with a solution of calcium hypochlorite was begun and has developed very rapidly. Later, chlorine gas was used directly for the same purpose.

The control of sparsely inhabited water-sheds, which minimizes the danger of contamination, is resorted to for the purpose of avoiding artificial purification at the points of consumption. Generally, even such water is also sterilized before delivery to the consumer.

The comparative cheapness of unfiltered water creates in the minds of people the idea that water should be as "free as air", and hence there results great waste in its use. The Municipal Engineer recognizes that artificially purified water is really a manufactured product, and economy of public funds demands curtailment of waste; therefore, the water meter was introduced to provide a measure for the water actually used, for which payment can be exacted.

Gas and Electricity.

The manufacture and distribution of gas and electricity and the lighting of the public streets are distinctly municipal problems. Whether the city owns and operates the plant or whether it purchases the commodity from a utility company, the services of a specialist are required, either to administer or to supervise the operations and to see that standards are maintained.

Sewerage Systems.

It is only within the past few decades that the public has come to a full realization that the health of the people very largely depends on a properly constructed and maintained sewerage system by which the liquid wastes of the community are promptly removed from their place of origin. Sewerage systems had their origin in America, as in Europe, in the construction of masonry culverts

to carry the small streams through the built-up portions of the city. There appears to have been little or no actual design of such culverts based on hydraulic principles, and when lateral extensions were made for conveying rain-water underground to the main culvert, the size of the lateral was not determined by the quantity of water to be carried, but rather by the diameter required for workmen to enter the culvert to remove deposits formed therein on account of insufficient velocities. In some cities the size of lateral sewers was arbitrarily fixed at 3 ft. in diameter.

In 1842, Lindley, an English engineer, laid out the sewerage system of Hamburg, and in his design provided for the continuous movement of the sewage by sufficient gradients and with periodic flushings.

Design of Sewers.

The design of American municipal sewerage systems on a comprehensive plan rather than piecemeal construction, was begun in 1855 in Chicago, Ill., by the late E. S. Chesbrough, Past-President, Am. Soc. C. E., followed shortly in Brooklyn by the late J. W. Adams, Past-President, Am. Soc. C. E., and in Boston by the late J. P. Davis, M. Am. Soc. C. E. These designs made use of empirical data obtained from European practice as to capacity and as to probable quantities of rain-water to be carried by the sewers.

The sewers of this early period, being developed from the old culverts, naturally were on the combined system, providing for the carriage of both sewage and rain-water in the same conduit. In England, as early as 1842, Chadwick had advocated the separate collection of sewage and storm-water, but it was not until 1880 that such a system was installed in a large city when the late Col. Waring designed the separate system of sewers for Memphis, Tenn.

The then National Board of Health sent Rudolph Hering, M. Am. Soc. C. E., to Europe to investigate sewerage practice, and his report thereon marks one of the turning points in sewer design in this country, as it placed such design on a scientific basis rather than rule-of-thumb. About the same time, the Burkli-Ziegler formula for determining the rate of run-off of rain-water was published and marked an advance in design, as it took into consideration the acreage and territorial slope of the drainage area tributary to the sewers. Subsequent modifications of this formula were devised for a number of American cities in efforts to obtain more accurate results.

Sounder principles of structural design were then introduced, proportioning the various parts of the sewer to the probable loads they would be required to carry and securing smoother interior surfaces.

What is known as the rational method of determining the maximum rate of run-off of storm-water is now quite generally used by competent designers, and many large cities maintain gauges for recording the rate of rainfall and the rate of run-off in the sewers. These data from existing sewers furnish the basis for future design.

The extension of the sewerage system of a city is now really necessary to its normal development. Modern living demands water supply and toilet conveniences in dwellings, and this requires facilities for the prompt removal of the sewage.

The broad problem of maintaining streams free from nuisance and in such a condition that they are suitable sources for public water supplies after purification, or for use in industry, is a State function, but the maintenance in a clean condition of streams flowing through or by a city is a municipal problem.

Development of Methods of Sewage Disposal.

Before considering the evolution of sewage disposal in America, it is advisable to note briefly the development of the art in England where the necessity for sewage treatment arose earlier than in other countries by reason of the dense population, numerous industries, and the relative smallness of the streams.

About the middle of the Nineteenth Century, in order to clean up the English towns, it became common practice first to utilize the existing storm drains and, later, to build new sewers for the conveyance of filth to the near-by watercourses, which naturally resulted in the serious pollution of the streams. Parliament, therefore, in 1857, created the Royal Sewage Commission, which, in its final report made in 1865, recommended the abatement of stream pollution by the application of sewage to land, that is, irrigation. Subsequently, the Second Royal Commission on Rivers Pollution, appointed in 1868, made its report in 1870, and recognized irrigation, intermittent filtration, and chemical precipitation as the then available processes of sewage treatment.

Meanwhile, many of the streams of Massachusetts had become polluted by the discharge into them of sewage and industrial wastes, and, in 1875, the State Board of Health made examinations of some of these rivers. Largely based on European practice, and bearing in mind the glacial formation common in Massachusetts, the Board recommended to inland towns the disposal of sewage by irrigation.

Fifty years ago the engineer in America had little choice of methods to prevent pollution of streams by city sewage. Worcester, about 1887, decided to use chemical precipitation to prevent the pollution of the Blackstone River caused by the discharge of crude sewage, and in the same year the Massachusetts State Board of Health began the now classic series of experiments on purification of sewage by intermittent filtration through beds of sand.

The effluent produced was clear, perfectly stable, and ofttimes of lower bacterial content than many well waters, and hence there arose in the minds of American engineers and sanitarians of that period the idea that "sewage disposal" meant the conversion of sewage into an effluent almost of drinking-water purity, and they utterly disregarded any subsequent purification in the receiving body of water through natural agencies.

The enormous quantities of watery, offensive sludge produced by the chemical precipitation of sewage, caused engineers to search for a process lacking this serious characteristic. There resulted the development of the contact bed in 1891 by the engineers of the London County Council, and thereafter this process became quite popular. Two years later, Corbett, at Salford, England, devised the trickling filter, which permitted higher rates of application than the contact bed and is to-day the most intensive, well established process for the oxidation of settled sewage.

The "activated sludge" process of sewage treatment was experimentally developed at Manchester in 1914. On this principle several installations were made subsequently in both England and America.

Sedimentation Tanks.

Simultaneously with the development in processes for oxidizing sewage, there was a corresponding evolution in the means for freeing the crude sewage of its suspended matter as a preliminary process. For many years it had been known that the organic solids in sewage were susceptible to decomposition and liquefication, but it was not until 1895 that Cameron, at Exeter, England, utilized these ideas on a large scale in a sedimentation tank intended to retain for a long time the deposited sludge.

The publication of the results accomplished in the Exeter septic tank led to the belief that at last a process had been found which practically eliminated the sludge problem that was such a serious one in chemical precipitation, and the result was that municipal engineers everywhere installed septic tanks, fondly trusting that they would be the remedy for their troubles. Extensive experience soon showed that the deposits were not all "digested", so that it was necessary to remove the sludge from these tanks, and that the decomposition of the sludge impregnated the effluent of the tank with offensive gases.

In 1904, Travis, at Hampton, attempted to overcome the latter difficulty by constructing a false bottom in the tank, which permitted about one-fifth of the raw sewage to flow through the lower sludge compartment. This resulted in a slight improvement in the quality of the effluent, but still produced offensive sludge. The complete separation of the settling sewage from the digesting sludge was devised by Imhoff about 1906, and tanks were constructed on this principle. The results of their operation were published in an English sanitary engineering paper in May, 1909, and in the following July an experimental tank on this principle, was put into operation by the City of Philadelphia in the Spring Garden Sewage Testing Station.

The intense interest and enthusiasm shown for this type of tank seemed for a time likely to reproduce the previous experience with the septic tank. Although some improperly designed tanks have not been successful, and other well-designed tanks have caused trouble, particularly in their early operation, due to foaming from gas vents, it appears to-day, particularly for large installations, to be the safest sedimentation tank, both from the point of view of removal of settleable material and production of minimum volumes of least offensive sludge.

There is a present tendency to accomplish the same results through the frequent removal of sludge from a plain sedimentation tank to a separate tank for subsequent digestion.

The application of a solution of calcium hypochlorite as a chemical germicide to the effluent of a municipal sewage works was first made in 1907 by Daniels at Red Bank, N. J., and has become common practice where the destruction of pathogenic bacteria is deemed necessary to protect sources of public water supplies or shell-fish beds.

Since 1877, it has been known that the chemical changes which sewage undergoes in its artificial treatment are produced through living organisms, but the

utilization of these same biological forces in natural waters has only been scientifically considered within recent years.

Notwithstanding the one-time almost universal practice of discharging raw sewage into streams, disposal by dilution was not considered as a method of treatment. The first attempt of an American municipality to dispose of sewage by dilution, as an avowed process for its purification, was made in the construction of the Chicago Drainage Canal. From that date there has been a gradual accumulation of knowledge as to the conditions which are necessary to provide for the inoffensive assimilation and complete oxidation of sewage through properly distributing it so as to bring about good diffusion throughout the cross-section of the receiving body of water. To-day, large sums of public money can often be saved by utilizing these natural powers in streams where conditions are such that the discharge of untreated or partly treated sewage will not constitute a menace to the public health.

Refuse Disposal.

The problem of the sanitary collection and disposal of the solid refuse of the city has been only recently taken up for scientific investigation by the Engineering Profession. Prior to that time all work of this kind was left to the householder and the man who collected garbage to feed to hogs. These methods were irregular and not dependable. Nuisance and serious inconvenience resulted, and the city authorities have been compelled, as a health measure, to assume the responsibility and make this work a part of the duty of the municipality.

The gathering of the city's wastes and its economical disposal or destruction is one of the latest but not least important problems of Municipal Engineering, for on its proper carrying out depends the health and comfort of the people.

Opportunity for Service.

As I review my experience of many years covering my connection in the practice of Municipal Engineering, I more fully appreciate the changes and improvements which have taken place in city government, whereby efficient and adequate public service has been provided to care for the rapidly increasing demands of the community. Our cities are more scientifically planned, have improved methods of transportation, are better paved and lighted, are provided with a more abundant supply of pure water, and the disposal of waste is properly cared for.

The Municipal Engineer has probably greater opportunity for service than those in other branches of the Profession, since he has to do with those problems which deal directly with the welfare of the people. If he uses his training and skill tactfully and wisely he will gain the confidence of the community and thus become a factor for good. By exercising foresight and initiative, he should be the leader in anticipating and directing the affairs and the development of the community.

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Paper No. 1475

THE FLOW OF LIQUIDS THROUGH SHORT TUBES

By Winslow H. Herschel,* Esq.

With Discussion by Messrs. Arthur C. Preston and Winslow H. Herschel.

Synopsis.

In hydraulics a short tube usually receives scant attention, but it has two extensive uses. The carburetor nozzle is a short tube which limits the supply of gasoline, and with most viscosimeters the time of flow through a short outlet tube is used as a measure of viscosity. It is proposed from an analysis of published data in regard to long tubes and orifices, supplemented by the small amount of data available on short tubes and by original tests, to study the laws of flow through short smooth tubes.

It is believed that the paper will prove to be of interest to hydraulic engineers. It contains, in addition to a summary of the literature of the subject, the following original features:

1.—Unpublished tests on an "inverted" Saybolt Universal viscosimeter.

2.—The determination of the kinetic energy correction for turbulent flow from the experiments of Couette. The value is not $\frac{v^2}{2g}$, as ordinarily assumed in hydraulics.

3.—The determination of the length of tube having the same law of flow in the stream line and in the turbulent régimes.

4.—Proof that Sorkau was in error in thinking his tests showed a lower value of Reynolds' criterion than that usually determined at the critical velocity.

Laws of Flow in Long Smooth Tubes.

a.—The Formula for Viscous Flow.

The term "viscous flow" may be used to designate "stream-line motion", or flow without turbulence, below the critical velocity. It is generally recognized that Equation (1), or its equivalent, applies to this régime of flow.†

$$\mu = \frac{\pi g d^4 t}{128 Q (l + \lambda)} \left(\gamma h - \frac{m \rho v^2}{g} \right) \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

* Associate Physicist, U. S. Bureau of Standards, Washington, D. C.

† Winslow H. Herschel, *Technologic Paper No. 112* (1919), p. 9.

where $\frac{Q}{t}$ = discharge, in cubic centimeters per second;

$\frac{\mu}{\rho}$ = kinematic viscosity, or viscosity, in poises, divided by density in grammes per cubic centimeter;

d = diameter of tube, in centimeters;

v = mean velocity in the tube, in centimeters per second;

g = acceleration of gravity = 981 cm. per sec.*;

$l + \lambda$ = effective length of tube, in centimeters, λ being the "Couette correction;"

m = coefficient of the kinetic energy correction;

h = average head, in centimeters, of liquid of density γ in grammes per cubic centimeter.

The last three quantities are not accurately known, and require a few words of explanation.

The necessity for the Couette correction is due to end effects not clearly understood. The average value of two determinations made by Couette is 2.7 d, if correction is made for a slight error pointed out by Brillouin.* Higgins† says it should be 0.82 d, but this theoretical value, which is due to Rayleigh, evidently applies only to submerged discharge. All the evidence indicates that λ is always positive and that it may be neglected in long tubes without serious error.

The most probable value of m appears to be 1.12, as calculated by Boussinesq‡ and approximately confirmed by Knibbs§ from a study of the experiments of Poiseuille and others. A value of unity is also frequently used, and it is a very common and persistent error to omit the kinetic energy correction entirely.

The average head may be taken as

$$\frac{h_1 - h_2}{\log_e \left(\frac{h_1}{h_2}\right)}$$

where h_1 is the initial, and h_2 the final, head. This value is approximate, however, since the kinetic energy correction was disregarded in its derivation.

Where, as is often the case, γ and ρ are equal, Q, d, l, and h are constant, and λ and m may be assumed to be constant or negligible, Equation (1) may be written:

$$\frac{\mu}{\gamma} = A\,t - \frac{B}{t} \qquad (2)$$

as used in viscosimetry, although under similar conditions, but with h variable, and with negligible variations in the kinematic viscosity, it may be written

$$h = ev + f\,v^2 \qquad (3)$$

No account is taken of the surface tension in Equation (1), so that the instrumental constants, A, B, e, and f, may vary slightly with the temperature of the liquid used, unless the discharge is submerged.

* M. Brillouin, "Leçons sur la Viscosité des Liquides et des Gaz," Vol. I (1907), p. 139.

† W. F. Higgins, Collected Researches, The National Physical Laboratory, Vol. XI (1914), p. 9.

‡ J. Boussinesq, Comptes Rendus, Vol. CXIII (1891), pp. 9–15 and 49.

§ G. H. Knibbs, *Journal and Proceedings*, Royal Soc. of New South Wales, Vol. XXIX (1895), p. 86.

To correct for surface tension, the net head $h - b$ may be used in Equation (3), where b is the height the liquid would rise in the tube by capillary action. Hagen* considered that b is equal to one-half this amount. If a correct value is used for b, and the resulting net head used for finding the constants in Equation (3), the results should be concordant. It was found in this way that the best concordance of results with different heads was obtained by giving b a value of about three-eighths of the capillary rise. The decrease in b from its theoretical value is probably due to the fact that water of ordinary purity, as used in hydraulic experiments, has a lower surface tension than the distilled water used in determining recorded values of surface tension.

b.—Reynold's Criterion for the Change from Viscous to Turbulent Flow.

The value of $\frac{v\,d\,\rho}{\mu}$ at the critical velocity at which turbulence begins will be called K. It is usually assumed to have a value of 2 000, and presumably Rummel† had this value in mind when he wrote:

"The values of w (velocity) obtained in carburetors, lie uniformly below Reynolds' critical velocity, although their maximum values are not very far therefrom."

That turbulence may occur, at least in airplane carburetor nozzles, is indicated by the value of Reynolds' criterion calculated from data concerning a 260-h.p. Mercedes engine with a "twin jet" carburetor.‡ If it is assumed that the kinematic viscosity of the fuel was 0.01, and that the velocity in the main and pilot nozzles was the same, Reynolds' criterion would have a value of 5 700.

For the Engler and Saybolt Universal viscosimeters‖ with outlet tubes comparable in length to carburetor nozzles, K was found to have a value of about 800. As will be shown later, K is a variable, and 2 000 is merely an approximate value which is frequently found to hold good for long tubes.

c.—Formulas for Turbulent Flow.

The most general equation which has been used for the flow in pipes was proposed by Rayleigh¶ and may be written

$$\frac{R}{l} = \rho\, v^2\, F\left(\frac{v\,d\,\rho}{\mu}\right) \qquad (4)$$

If $\gamma = \rho$, this reduces to

$$\frac{h}{l} = \frac{4\,v^2}{g\,d}\, F\left(\frac{v\,d\,\rho}{\mu}\right) \qquad (4a)$$

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* G. H. L. Hagen, Abhandlungen der Königlichen Akademie Wissenschaft, Berlin (Math.), for the year 1854, p. 53.

† K. Rummel, *The Horseless Age*, Vol. XXXV (1915), p. 511; also C. E. Lucke, 2d Annual Report, National Advisory Committee for Aeronautics (1917), p. 466.

‡ *Aerial Age Weekly*, Vol. VI (1917), p. 115.

‖ Winslow H. Herschel, *Proceedings*, Am. Soc. for Testing Materials, Vol. XVII, Part II (1917), p. 551.

¶ Lord Rayleigh, Report of the British Advisory Committee for Aeronautics, Vol. I (1909–10), p. 38.

This formula has been found to apply to steam* as well as to water and oil.† Using the data of Stanton and Pannell, Lees‡ found that

$$\frac{R}{\rho\, v^2 - a}\left(\frac{v\, d\, \rho}{\mu}\right)^q = k \qquad (5)$$

in which R is the frictional resistance, in dynes per square centimeter, on the inner surface of the tube. To get the relation between R, h, and the pressure gradient, i, as used by Couette‖ it may be noted that the total force in dynes acting on a section of diameter, d, is

$$(h\, \lambda\, g)\, \frac{\pi}{4}\, d^2$$

and this, divided by the rubbing surface, $\pi\, d\, l$, gives R, the loss per unit area, as equal to

$$\frac{\gamma\, h\, g\, d}{4\, l} = \frac{d}{4}\, i$$

The constant, a, may be omitted if the range in values of vd is not excessive, and then Equation (5) may be written:

$$\frac{h}{l} = \frac{C\, \rho\, v^{2-q}\left(\frac{\mu}{\rho}\right)^q}{\gamma\, g\, d^{1+q}} \qquad (6)$$

where $C = 4\, k$. For a given liquid at constant temperature, assuming $q = 0$, Equation (6) reduces to the well known Chezy formula. Equation (6) appears to be the simplest equation which takes into account variations in viscosity, and which will serve for calculating the flow of liquids in pipes, with moderate velocities, in the régime of stable turbulent flow.

In viscous flow, q is equal to unity, and Equation (6) reduces to,

$$\frac{h}{l} = \frac{C\, v}{\gamma\, g\, d^2} \qquad (7)$$

which may be derived from Equation (1) if the kinetic energy and Couette corrections are neglected.

The exponents in Equation (6) may be determined separately, by experiments in turbulent flow, without reference to the indicated relationship between them. Tests of Stanton and Pannell, and of Couette with water, were used, as shown in Fig. 1, to find the exponent of v. The variations of temperature were so slight that no attempt was made to correct for variations in kinematic viscosity.

Couette used for each series of tests a pair of tubes of the same diameter, but of different length, assuming, as would generally be the case, that the difference in the loss of head in the two tubes was equal to that in an imaginary tube, without end effects, and of a length equal to the difference in length of the actual tubes. since, however, the régime of flow is determined to some extent by the length of tube, and the loss of head due to end effects is of different value in the two régimes,

* C. H. Lander, *Proceedings*, Royal Soc., Vol. XCII (1916), p. 337.

† T. E. Stanton and J. R. Pannell, Collected Researches, National Physical Laboratory, Vol. XI (1914), p. 295; J. R. Pannell, Vol. XIII (1916), p. 259.

‡ C. H. Lees, *Proceedings*, Royal Soc., Vol. XCI (1914), p. 49.

‖ M. Couette, *Annales de Chimie et de Physique*, Vol. XVI (1890), p. 481.

it might occur in some cases that the end effects would not be eliminated by this method.

Only those tests of Stanton and Pannell were taken in which Reynolds' criterion was below 20 000, except for the pipe with a diameter of 0.361 cm., for which the value of 20 000 was exceeded for most of the tests.

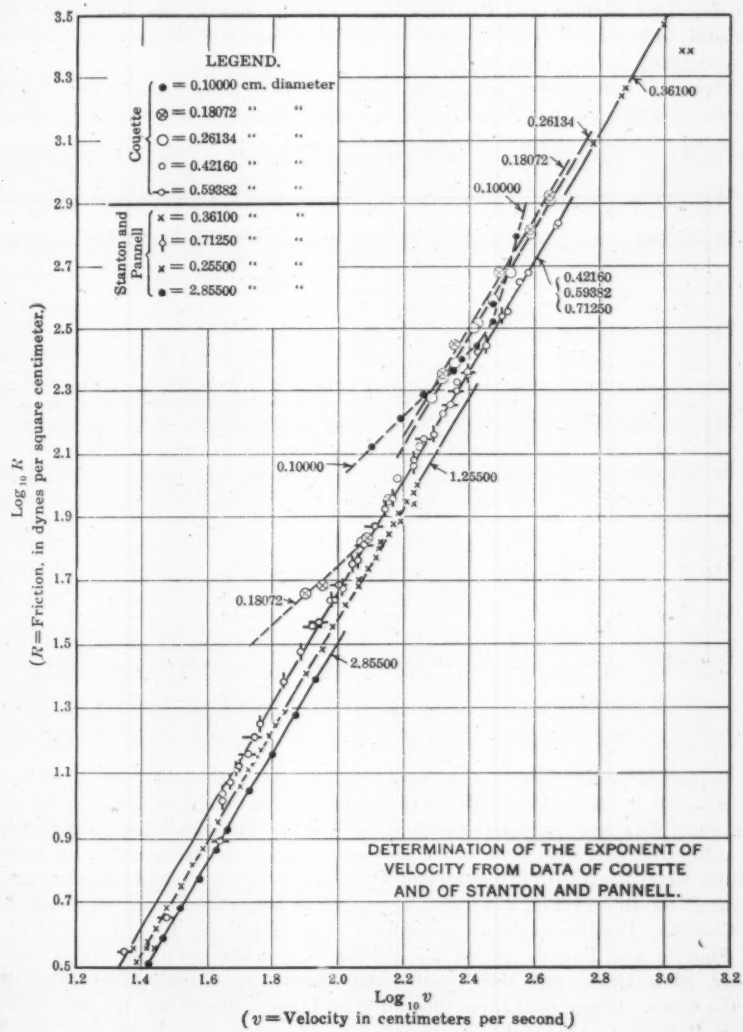

FIG. 1.

It will be seen that, in general, Fig. 1 shows straight parallel lines, making with the axis of abscissas an angle the tangent of which is 1.75, which is the value of $2 - q$. In some cases, especially noticeable for the smallest tube, the tests at the lowest velocities show the characteristic 1 : 1 slope of viscous flow. For slightly higher velocities the discordant results of the unstable régime may be noticed. The fact that Stanton and Pannell's pipes and the largest tube of Couette were of metal, while Couette's other tubes were of glass, does not appear to have affected the constancy of the exponent of v.

There are few reported tests showing a lower value than 1.75, and it is safe to assume that for finished metal tubes the smoothness will be such as to cause a value not exceeding and probably not less than this.

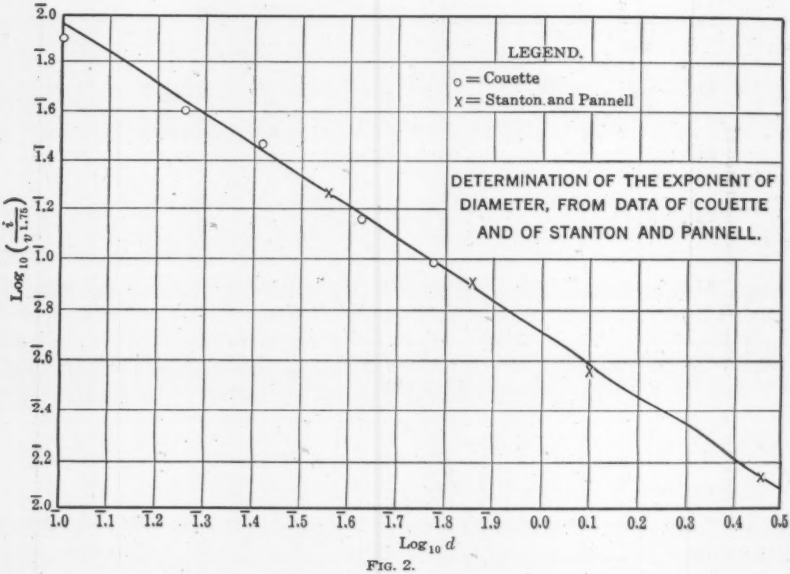

FIG. 2.

In finding the exponent of d, care must be taken that all tubes used in the calculations are equally smooth, as shown by the constancy of the exponent of v. The same data were used for Fig. 2 as for Fig. 1. In calculating average values of ordinates, individual tests which showed a marked departure from the average results were discarded. Stanton and Pannell's tests were given more weight in drawing the line in Fig. 2, because they showed better agreement than did those of Couette. The line as drawn indicates a value of 1.25 for $1 + q$.

Stanton and Pannell varied the kinematic viscosity by using air as well as water, while Hagen and Mair* varied the viscosity by changing the temperature of the water. Hagen used three tubes of sheet brass, soldered into the desired form, while Mair used a brass tube.

* J. G. Mair, *Minutes of Proceedings*, Inst. C. E., Vol. LXXXIV, Pt. 2 (1886), p. 424.

In Fig. 3 the uncorrected ordinates,

$$\frac{C\,\rho\left(\frac{\mu}{\rho}\right)^q}{\gamma\, g}$$

plotted against the kinematic viscosity, give for Mair's tests a straight line making with the axis of abscissas an angle the tangent of which is 0.274. This same

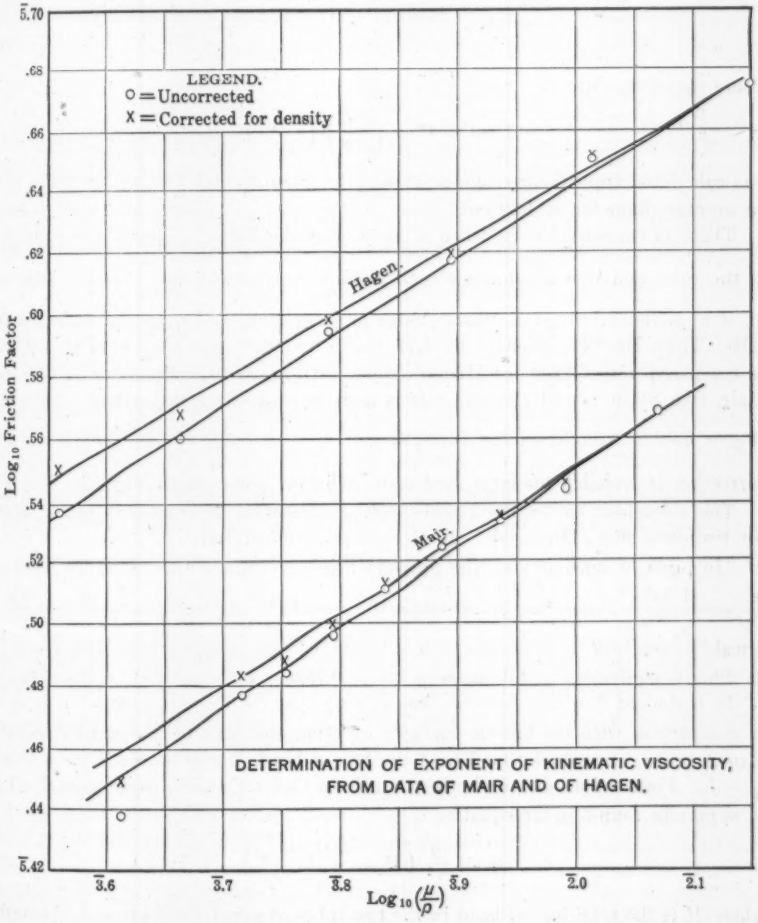

FIG. 3.

value of q was obtained by Knibbs*, who overlooked the necessity of correcting the ordinates by dividing by ρ. The corrected value of q is 0.24, which agrees a

* G. H. Knibbs, *Journal and Proceedings*, Royal Soc. of New South Wales, Vol. XXXI (1897), p. 338.

little more closely with Knibbs' value of 1.78 for the exponent of v (calculated from Mair's data) than it does with Mair's own value of 1.795.

Similarly, the corrected line on Fig. 3 for Hagen's tests gives a value of q equal to 0.222, which agrees exactly with Hagen's average value of 1.778 for the exponent of v.

It may be concluded from Figs. 1 to 3 that q is equal to 0.25 for finished metal tubes, and that Equation (6) may be written:

$$\frac{h}{l} = \frac{0.1580\, v^{1.75} \left(\frac{\mu}{\rho}\right)^{0.25}}{\gamma\, g\, d^{1.25}} \qquad (8)$$

where the coefficient

$$0.1580 = C = \left(\frac{4\,R}{\rho\, v^2}\right)\left(\frac{v\, d\, \rho}{\mu}\right)^{0.25}$$

was calculated from the average of 78 tests by Stanton and Pannell on tubes with an average diameter of 1.30 cm.

There is considerable evidence to show that the factor C depends on the size of the tube, and that roughness should perhaps be measured by $\frac{h'}{d}$ rather than by h', if h' is the height of a minute ridge or projection, and d the diameter of the tube. Thus, Mair's tests on a tube 3.78 cm. in diameter gave a value of $C = 0.1051$ as compared with 0.1203 for Hagen's tubes with an average diameter of 0.43 cm. Mair, like Stanton and Pannell, eliminated the end effects experimentally, while Hagen used a kinetic energy correction of $\frac{0.866\, v^2}{g}$. As will be seen later, this correction is probably too large, and resulted in too low a value for C.

The coefficient, m, of the kinetic energy correction is ordinarily taken as 0.5 for turbulent flow. Reynolds* states, without explanation,

"In pipes of sensible size, the pressure necessary to start the fluid lies between $\frac{v^2}{2\,g}$ and $\frac{1.505\, v^2}{2\,g}$ according to whether the mouthpiece is trumpet-shaped or cylindrical."

This is equivalent to taking m as 0.5 or 0.7525.

In turbulent flow the velocities are so high that the Couette correction is small in comparison with the kinetic energy correction and may be disregarded without serious error. In calculating Fig. 1 the loss of head, h, was obtained for a length $l_1 - l_2$. From values of h and other data from Couette's tests, experimental values of m may be found by the equation

$$m = \frac{g}{v^2}\left(H - \frac{h\,(l_1 + l_2)}{(l_1 - l_2)}\right) \qquad (9)$$

where H is the total loss of head in the two tubes of equal diameter and of lengths, l_1 and l_2, respectively.

Table 1 shows values of m for the turbulent régime calculated from Equation (9), omitting values greater than unity which probably indicate viscous flow.

* O. Reynolds, *Philosophical Transactions*, Royal Soc., Vol. CLXXIV, Pt. 3 (1883), p. 981.

It will be seen that Table 1 agrees fairly well with Reynolds' value of 0.7525 for tubes with cylindrical ends. The graphical method used for finding the values of m in the last column will now be considered.

TABLE 1.—VALUES OF m FOR TURBULENT FLOW, FROM COUETTE'S DATA.

Diameter, in centimeters.	Length, in diameters.	m from Equation (9).	Number of tests averaged.	m, found graphically.
0.1	52.7	0.769	..	.....
0.1	158.5	0.769	1	.....
0.18072	40.1	0.624	..	0.658
0.18072	125.2	0.624	5	0.724
0.26134	58.8	0.778	..	0.814
0.26134	106.5	0.778	10	0.818
0.4216	23.2	0.936	..	0.911
0.4216	164.0	0.936	..	0.771
0.4216	44.8	0.936	..	0.910
0.4216	119.1	0.936	10	0.811
0.59882	74.8	0.586	..	0.616
0.59882	222.0	0.586	14	0.696
	Average.....	0.731.....		0.773

HERSCHEL DIAGRAM FOR STUDYING LAWS OF FLOW.

As previously shown,* if Reynolds' criterion is plotted against $\frac{\mu'}{\mu}$ (where μ' is the viscosity as obtained from Equation (1) if the kinetic energy and Couette corrections are neglected), then with viscous flow,

$$\tan \theta = \frac{32\, l}{m\, d} \qquad (10)$$

where θ is the angle between the graph and the axis of abscissas.

In the viscous régime the graphs, for tubes of different lengths, intersect at a point on the axis of abscissas at a distance of $\frac{l+\lambda}{l}$ from the origin, and (as would appear from Fig. 4 to be approximately the case) it may be assumed that the graphs of the turbulent régime intersect at the same point when extended. Then, from Equation (8)

$$\cot \theta = \frac{0.1580}{32 \left(\frac{v\, d\, \rho}{\mu}\right)^{0.25}} \left(\frac{l+\lambda}{l}\right) + \frac{m\, d}{32\, l} - \frac{\mu}{v\, d\, \rho} \left(\frac{l+\lambda}{l}\right) \qquad (11)$$

The last column of Table 1 was calculated from Equation (11), using a value of 8 000 for Reynolds' criterion, neglecting the Couette correction, and taking values of cot θ from Fig. 4. It was found by successive approximations, with the help of Equation (11), that the most probable average value of m for Hagen's three tubes was 0.75, giving a value of 0.1840 for C.

Since, with an infinitely long tube, or tests such as those of Stanton and Pannell in which end effects are eliminated experimentally, there is no need of the kinetic energy correction, $\frac{\mu'}{\mu}$ will equal unity for all velocities up to the critical.

* W. H. Herschel, *Technologic Paper No. 112* (1919), p. 16.

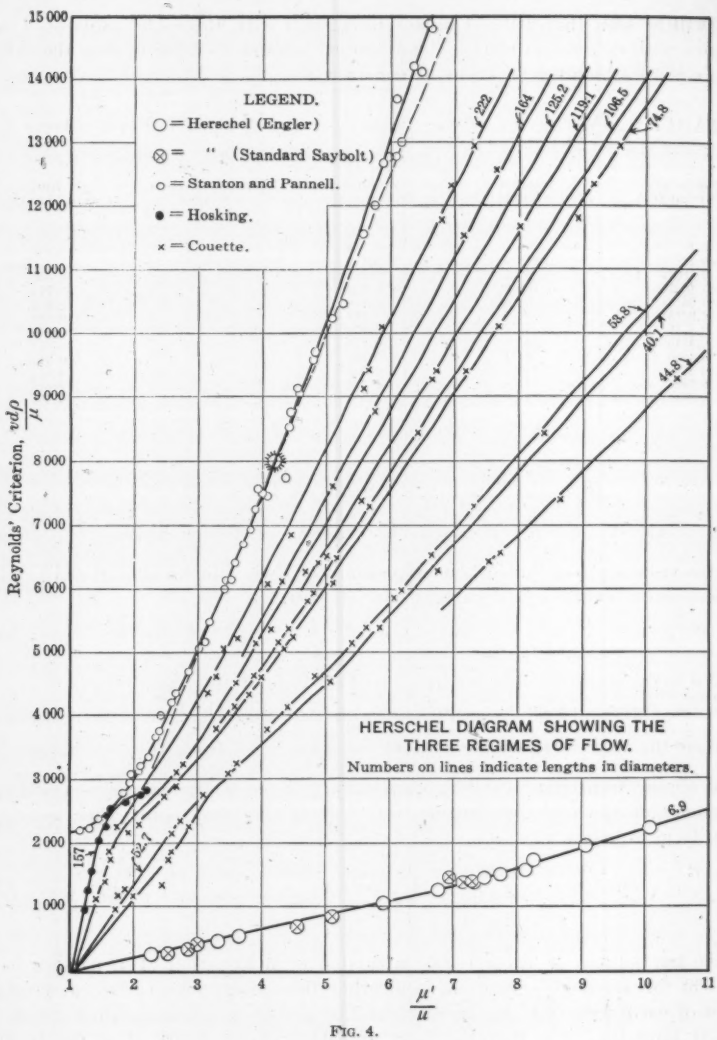

FIG. 4.

The departure of the graph for Hosking's test* from this value is due to the end effects. In the turbulent régime, if the end effects are eliminated, Equation (11) indicates that the graph will not be straight, as cot θ varies with the velocity. Taking Reynolds' criterion as 8 000, and neglecting the Couette correction, the calculated value for cot θ for Stanton and Pannell's tests was found to be 0.000397,

* R. Hosking, *Philosophical Magazine*, Vol. XVIII (1909), p. 260.

which, as shown by the dotted line, Fig. 4, agrees with the tests in the neighborhood of the ordinate of 8 000 almost as well as the curved line.

Couette made use of Equation (3), which gives a straight line on the Herschel diagram, for both the viscous and the turbulent régimes of flow, and Fig. 4 shows that he was justified, since, when the end effects are not eliminated, any curvature of the graph, if it exists, is small compared with the experimental error.

As shown by Equation (11), cot θ will increase as the length of the tube in diameters decreases, and this is plainly seen in Fig. 4. For a certain length of tube the slope of the graph will be the same for turbulent as for viscous flow. Taking Reynolds' criterion as 2 000, it was calculated that this length would be 48.5 diameters, which agrees fairly well with the fact that Couette's tube, 40.1 diameters in length, shows approximately the same slope in the viscous and turbulent régimes, the intermediate, unstable régime being indicated by a bulge in the graph.

It will be noted that tests on the Engler and standard Saybolt Universal viscosimeters* give practically the same graph, since both instruments have outlet tubes of approximately the same smoothness and ratio of length to diameter. On the other hand, the transposed position of the graphs for the two shortest tubes of Couette would appear to indicate that one was rougher than the other.

The Value, K, of Reynolds' Criterion at the Critical Velocity.

Hagen, who was perhaps the first to observe the three distinct régimes of flow, gives data from which it may be calculated that Reynolds' criterion at the critical velocity has a value, K, of 2 630, and that the unstable régime continues until the velocity is 1.22 times the critical. Reynolds locates the beginning of stable turbulent flow at 1.325 K, while Preston,† who calls this the "post critical velocity", places it at 1.41 K.

The lowest value of K quoted by King‡ is 180, as found theoretically by Orr‖, but as noted by Carothers,¶ when the diameter is taken in inches instead of in feet, Orr's value is 2 160, which is a fair check on the usually accepted value of 2 000. Coker and Clement** used a brass pipe, 183 cm. in length and 0.9599 cm. in diameter, and although they did not actually determine the critical velocity, it may be calculated from their data that K was between 4 340 and 4 940, or higher than any other value quoted by King. They attribute this high value to "complete absence of vibration in the tank, which was founded on rock, and also the freedom of the water from sediment." If such causes as these materially change the value of K, it is futile to expect agreement between theory and experiment, except perhaps for some limiting case.

Sorkau†† used a glass tube 5 cm. long and 0.0423 cm. in diameter, and found an average value of 414 for K with water, which is the lowest value reported as the result of experiment.

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* W. H. Herschel, *Proceedings*, Am. Soc. for Testing Materials, Vol. XVIII, Part 2 (1918), p. 364.

† A. C. Preston, *Chemical and Metallurgical Engineering*, Vol. XXIII (1920), p. 609.

‡ L. V. King, *Philosophical Magazine*, Vol. XXIX (1916), pp. 322, 338.

‖ W. M'F. Orr, *Proceedings*, Royal Irish Academy, Vol. XXVII A, (1907), p. 69.

¶ S. D. Carothers, *Proceedings*, Royal Soc., Vol. LXXXVII (1912), p. 158.

** E. G. Coker and S. B. Clement, *Philosophical Transactions*, Royal Soc., Vol. CCI (1903), p. 52.

†† W. Sorkau, *Physikalische Zeitschrift*, Vol. XVI (1915), p. 98.

In Fig. 5, calculated from Sorkau's data, the break in the graph shows that K is 7 000, while the slope of the straight part of the graph indicates viscous flow, since from Equation (10), m is 0.996. Thus, Sorkau's value of 414 is seen to be in error, and the lowest value of K obtained experimentally for long tubes is about 1 900, as found by Reynolds,* Morrow† and by Barnes and Coker,‡ while the lowest value for short tubes is 800, previously referred to. Brillouin recognized that K depends on the length, as well as on Reynolds' criterion for very short tubes, but gives no numerical values.

Sorkau, Brillouin, and Lewis§ propose to locate the critical velocity at the point of intersection of graphs calculated from equations similar to Equations (1) and

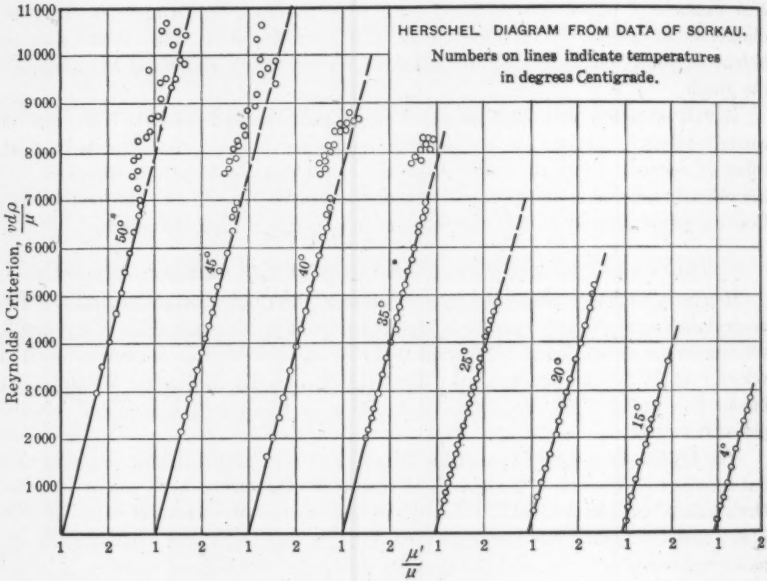

FIG. 5.

(6). This rests on the untenable assumption that there is no intermediate, unstable régime between that of viscous and of stable turbulent flow. Fig. 4 shows that this is certainly not the case for long tubes.

TORRICELLI'S EQUATION FOR FLOW THROUGH AN ORIFICE.

The discharge through an orifice is usually calculated by the equation:

$$v = c\sqrt{2gh} \qquad (12)$$

where the average head, h, is obtained by the equation:

$$h = \left(\frac{\sqrt{h_1} + \sqrt{h_2}}{2}\right)^2 \qquad (13)$$

* O. Reynolds, *Philosophical Transactions*, Royal Soc., Vol. CLXXXVI, Pt. 1 (1895), p. 124.
† J. Morrow, *Proceedings*, Royal Soc., Vol. LXXVI (1905), p. 211.
‡ H. T. Barnes and E. G. Coker, *Proceedings*, Royal Soc., Vol. LXXIV (1904), p. 355.
§ W. K. Lewis, *Journal of Industrial and Engineering Chemistry*, Vol. VIII (1916), p. 267.

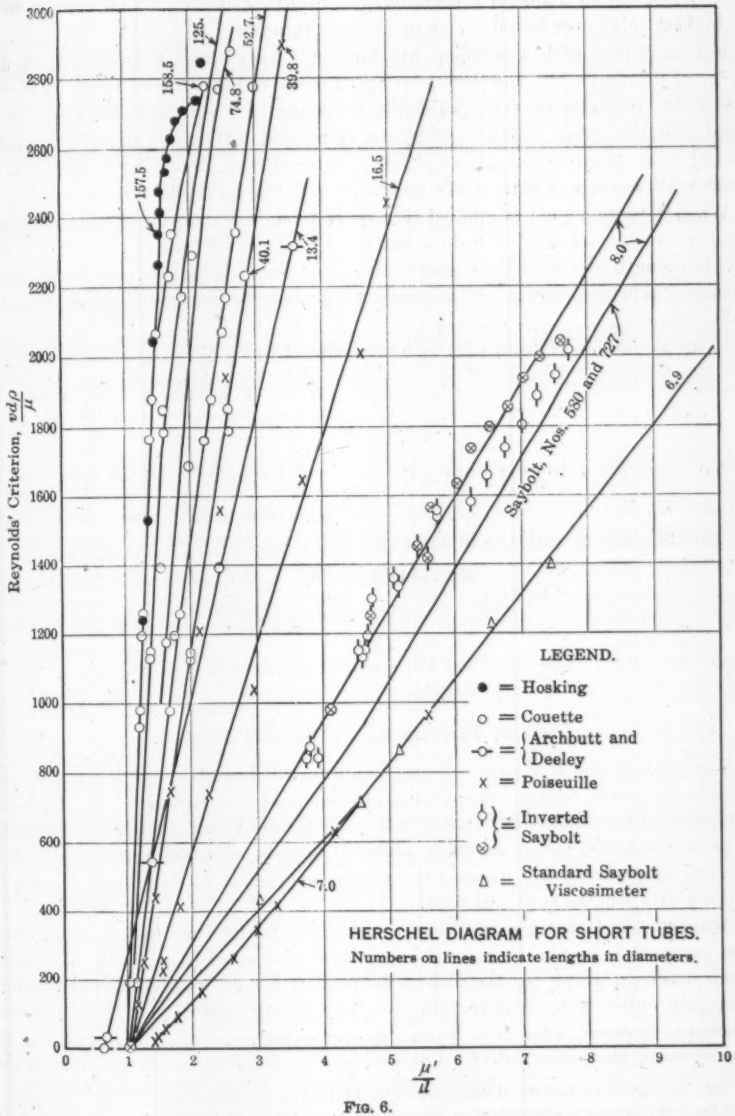

FIG. 6.

which must be regarded as approximate because it is based on the assumption that c, the coefficient of discharge, is constant. Although c has a fairly constant value for high heads and large openings, it is known that it increases as the diameter decreases or as the head decreases. Hachette,* for example, found with an

* J. N. P. Hachette, *Annales de Chimie et de Physique*, Vol. III (1816), p. 80.

orifice 0.055 cm. in diameter a coefficient of discharge of 0.77, which appears to be the highest value ever found for an orifice in a thin plate.

For an orifice with a conoidal mouthpiece, Unwin* found coefficients of discharge of 0.9418, 0.9740, and 0.9871 for water with temperatures of 15.6°, 54.4°, and 87.8° cent., respectively, thus indicating a change in c with the viscosity. Buff† found a similar result with an orifice in a thin plate, 0.47 cm. in diameter, under a head of from 54 to 2.9 cm., the coefficient of discharge increasing from 0.6535 to 0.6918 with water, and from 0.6630 to 0.7278 with ethyl alcohol.

When Equation (12) is applied to a short tube, it is even more unsatisfactory, because c varies not only with diameter, head, and viscosity of the liquid, but also with the length of tube. With short tubes, c is found to increase as the diameter decreases, as with orifices, but, contrary to the law for orifices, c increases as the head increases.‡

From Equations (1) and (12) and the definition of μ':

$$\mu' = \frac{\gamma\, d^2 \sqrt{g\, h}}{32\, c\, l \sqrt{2}} \qquad (14)$$

so that when h is zero (and, consequently, v and Reynolds' criterion are zero), $\frac{\mu'}{\mu}$, will also be zero, and the graph, therefore, will pass through the origin. Then, if θ has the same meaning as in Equation (10):

$$\tan\theta = \frac{\text{Reynolds' criterion}}{\frac{\mu'}{\mu}} = \frac{64\, c^2\, \rho\, l}{\gamma\, d} \qquad (15)$$

Hence, for a given tube or orifice, tan θ will be constant if c is constant.

Laws of Flow in Short, Smooth Tubes.

It is evident that if a tube is long enough to run full at the outlet, any further increase in length would increase the friction and thus decrease the discharge. The length of maximum discharge is ordinarily assumed to be about three diameters for moderate heads, but as shown by Rummel's§ tests with very low heads, such as are used with carburetor nozzles, the maximum coefficient of discharge may be obtained with lengths as low as 0.5 d.

Although the Couette correction may be neglected for long tubes, except where great accuracy is required,‖ it is of importance with short tubes. Fig. 6 contains, among others, a graph for the Coleman-Archbutt 100-cu. cm. viscosimeter, which apparently indicates a negative value for the Couette correction. There is reason to believe, however, that this is due to inaccuracy in the data regarding the dimensions of the outlet tube.¶ The excessive positive value of the Couette cor-

* W. C. Unwin, *Philosophical Magazine*, Vol. VI (1878), p. 285.

† H. Buff, *Annalen Physikalische Chemistrie* (2), Vol. XVI (1839), p. 240; also U. S. Bureau of Standards *Circular No. 70* (1917), p. 231.

‡ See E. G. Hopson, *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 572; P. S. Tice, Report No. 49 (1920), National Advisory Committee for Aeronautics, p. 16.

§ K. Rummel, *The Horseless Age*, Vol. XXXV (1915), p. 508, translated from *Der Motorwagen*, Berlin, 1906.

‖ *Proceedings*, Am. Soc. for Testing Materials, Vol. XVIII, Part 2 (1918), p. 385.

¶ L. Archbutt and R. M. Deeley, "Lubrication and Lubricants", pp. 169, 181 (1912).

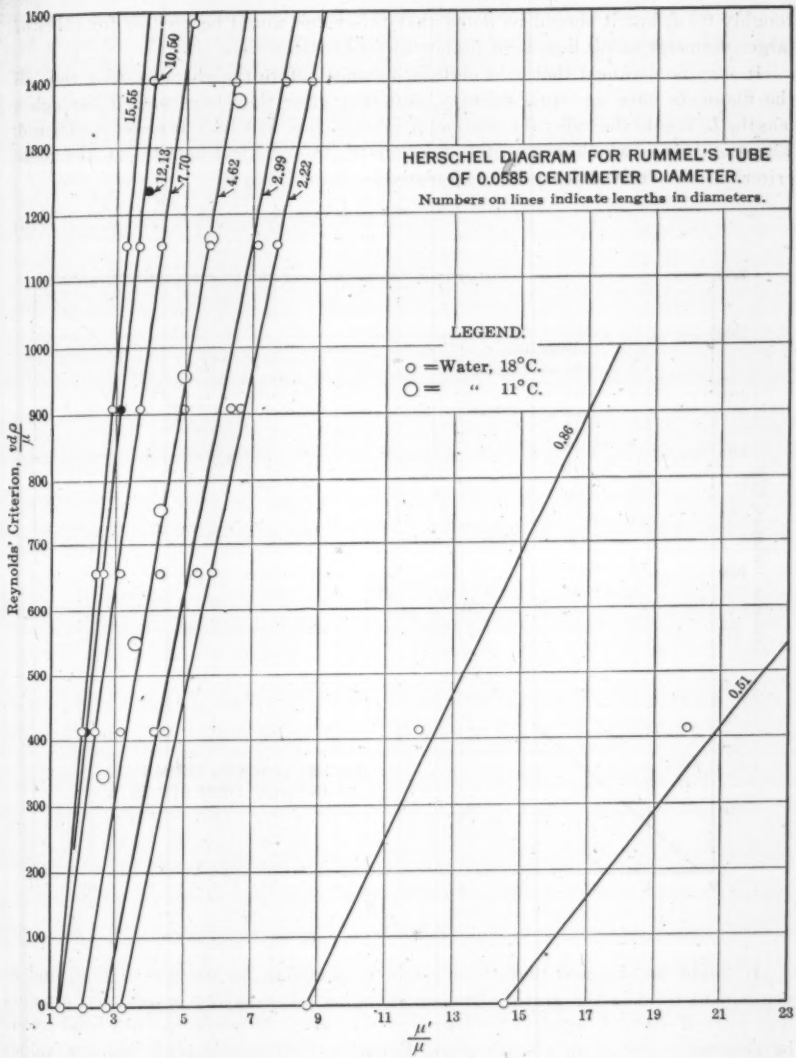

FIG. 7.

rection indicated by Poiseuille's shortest tube is probably likewise due to an error in length, as he states that it is uncertain.*

With viscosimeters discharging into the air, the value of λ was found to be

* J. L. M. Poiseuille, *Mémoires* de l'Institut Savants Etrangers, Vol. IX (1846), p. 433.

roughly 0.5 d, but it should be noted that this value would be greater for tubes of larger diameter using liquids of higher surface tension than oil.

It may be assumed that at a certain distance before the entrance to a tube all the filaments have an equal velocity, and that after they have passed through a length, L, inside the tube, the régime of viscous flow will have become established. Equation (16), which shows the relation of this critical length to Reynolds' criterion, was derived from data calculated by Boussinesq:*

$$\frac{v\,d\,\rho}{\mu} = 15.4\,\frac{L}{d} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

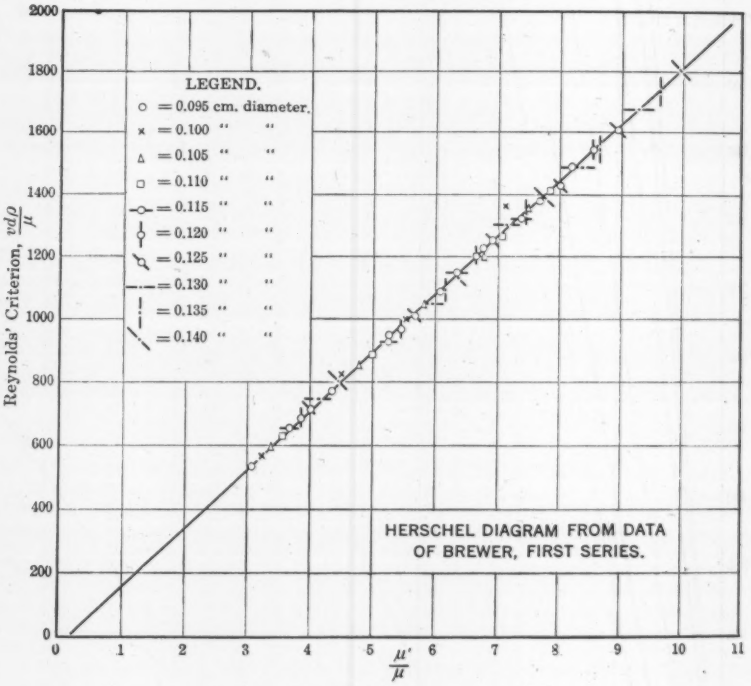

FIG. 8.

It might be expected that there would be a change in the law of flow, and a consequent bend in the graph when, on changing the viscosity or velocity, a point was reached where L equalled the length of the tube. Although no such bend could be detected in work on viscosimeters, Equation (16) may explain the low value of m found with very short tubes. For since the value of 1.12 was based on Equation (16), it is to be expected that if the length of tube is less than L, the acceleration would be incomplete, and the coefficient of the kinetic energy correction would be less than 1.12. As it would be less than unity with turbulent flow, it would be difficult to distinguish one régime from the other by means of the

* J. Boussinesq, Comptes Rendus, Vol. CXIII (1891), p. 50.

value of m, as may be done with long tubes. Apparently, 0.5 is the limit approached by m as the length approaches zero, for either turbulent or viscous flow, since $m = 0.5$ if the velocity is the same at all points on a diameter.

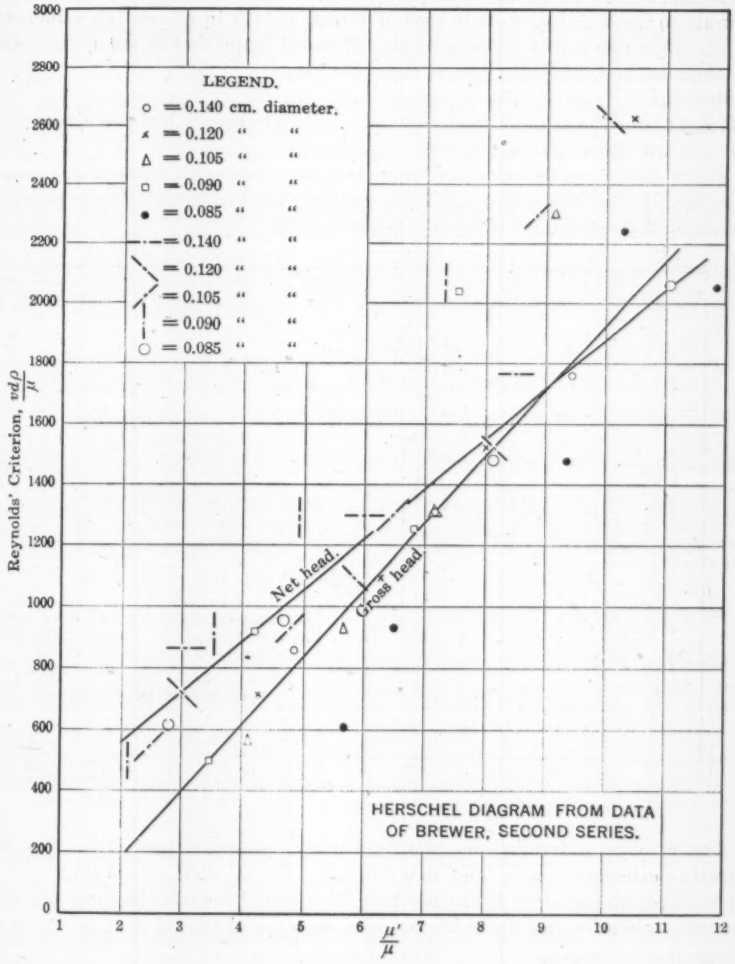

FIG. 9.

EXPERIMENTAL DATA ON FLOW THROUGH CARBURETOR NOZZLES.

In Rummel's tests the head was kept constant during any one test, but varied in different tests from zero to 65 cm. He used water in tubes discharging vertically upward, so that a water cap was formed at the outlet which vitiated his results.

Fig. 7 shows tests with a tube 0.0585 cm. in diameter, and tubes of three other sizes gave similar diagrams. It will be seen that in general the shorter the tubes the greater the apparent value of the Couette correction. Since the length of tubes is less than L, and the régime of flow is uncertain, it seemed more simple and equally accurate to use Equation (10) instead of Equation (11) in calculating values of m for Fig. 10, which will be described later. Rummel found that m was in some cases less than 0.5, and attributed this to the water cap.

Brewer's tests were made with gasoline.* In the first series, constant heads of 3, 6, 12, and 15 cm. were used. In the second set the gasoline discharged vertically upward into a reduced pressure.

Fig. 8 shows results calculated from his first series with tubes five diameters in length. It was assumed that $\rho = \lambda$, and that the kinematic viscosity was 0.01. The slope of the graph gives for m a value of 0.862.

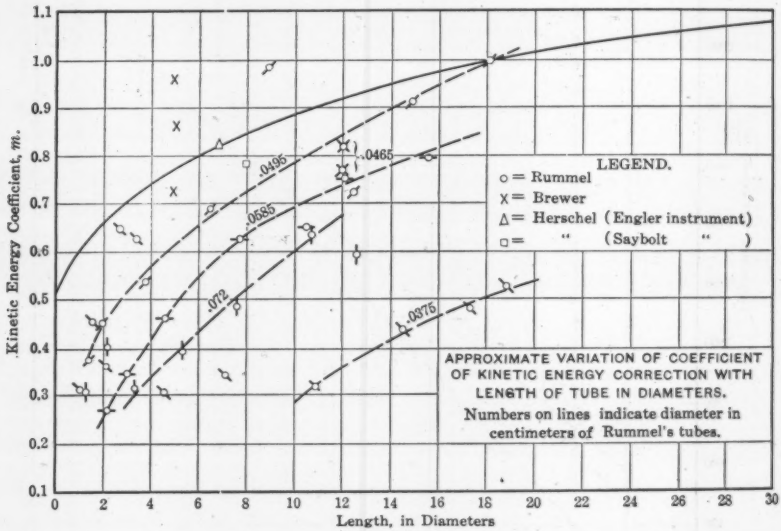

FIG. 10.

Fig. 9 shows a Herschel diagram for Brewer's second series. If the total head is used in calculation, m is equal to 0.720, but if the calculations are made with the net head remaining after deducting the head required to start the flow, m is equal to 0.960. There is thus no indication of a decrease in the value of m due to discharge into a vacuum. Fig. 9 gives some indication of a critical velocity at a value of Reynolds' criterion of 2 000.

The full line in Fig. 10 has been sketched from the information available to show variations in m with change in the ratio of length to diameter. It was assumed that Rummel's values were too low, on account of the water cap, and that his longest and largest tubes, in which the effect of the water cap and of surface tension would be proportionally the least, would give the most nearly correct results.

* R. W. A. Brewer, "Carburetion", pp. 48–72.

Discharge Under a Constant Head.

No formula is available which will give the correct average head except when c is constant or the kinetic energy correction is negligible. It was found by tests with viscosimeters that m was increased if the variation in head was decreased by using reduced volumes of discharge.

TABLE 2.—Times of Discharge for Saybolt Universal Viscosimeter No. 580, Inverted, with Abnormal Volumes Put in and Discharged.

Volume put in, in cubic centimeters.	Volume discharged, in cubic centimeters.	Temperature, in degrees Centigrade.	Time, in seconds.	
100	100	10	64.1	
200	200	10	124.7	
60	60	11	39.9	
60	60	15	37.9	
60	60	20	37.2	
60	60	20	36.7	
60	60	20	42.9	Fell free
100	100	20	70.1	Fell free
200	200	20	134.0	Fell free
200	200	20	118.8	
100	100	20	61.3	
400	400	20	224.1	
200	60	20	33.7	
400	60	20	31.2	
600	60	20	30.0	
600	60	20	29.1	
800	60	20	27.3	
800	60	20	27.4	
1 000	60	20	26.8	
1 200	60	20	25.5	
1 400	60	20	24.4	
1 600	60	20	23.4	
1 800	60	20	22.4	
2 000	60	20	21.7	
2 200	60	20	21.0	
Elevation Checked After Each Test of Ten Runs.				
200	60	20	33.9	
600	60	20	29.9	
600	60	20	29.0	
800	60	20	27.1	
1 000	60	20	25.9	
1 200	60	20	24.4	
1 400	60	20	23.5	
1 600	60	20	22.8	
1 800	60	20	21.8	
2 000	60	20	21.2	
2 200	60	20	20.7	
60% Sugar Solution.				
200	60	35	172.5	
200	60	40	144.7	
200	60	45	123.3	
200	60	50	108.9	

The attempt was made with the Saybolt viscosimeter to find the effect of a more constant head, by inverting the container, and using the bath to contain the liquid to be tested, before it was discharged through the inverted outlet tube. This attempt was only partly successful, because with the exception of the three tests marked "fell free" in Table 2, the liquid trickled down the walls of the inverted container, and no method of manipulation was discovered to prevent this. In the cases where comparison could be made, the time of discharge was greater when the jet fell free,

and in this latter case the time more nearly approached that for the normal use of the instrument.

In some of the tests with the inverted outlet tube, the volume of water in the container (or rather, in the bath used as a container) was checked after each test, and these tests are given greater weight in drawing the graph for the inverted instrument, Fig. 6. The point which determines the lower end of the graph was obtained with 60% sucrose solution, the other points being determined with water.

It will be noted that the graph for the inverted Saybolt viscosimeter is parallel to the graph in the turbulent régime for the instrument run in the usual manner, and indicates that m equals 0.785 in both cases. The absence of a bend in the graph would seem to show that all the tests were in the same régime of flow, and the value of m would indicate the turbulent régime. That the inversion of a tube may have a marked effect on the flow has been noted by Ronceray,* but in the present instance it seems possible that turbulence was caused by the increase in the size of the container. This would be in accord with previous tests in which it was observed to be very difficult to determine the time of flow with water in the Engler viscosimeter, on account of turbulence due to swirling, although no trouble was experienced with the Saybolt viscosimeter in its normal position.

Conclusions.

1.—The results of this investigation show that for viscous flow in long tubes Equation (1) is the most satisfactory formula, values of the Couette and kinetic energy corrections being known with sufficient accuracy. The value of the Couette correction probably lies between zero and three diameters, while m has a most probable value of 1.12.

2.—The formula for turbulent flow is more complex because the friction factor C must be included. The best formula is Equation (6), for which values of C and q have been given. This equation does not contain either the Couette or the kinetic energy corrections, of which the former may be neglected, while the latter should be subtracted from h to give the head available for overcoming friction and viscosity. The value of m in turbulent flow is approximately 0.75.

3.—The value of Reynolds' criterion helps to determine whether the critical velocity has been passed, and whether Equation (1) or Equation (6) should be used, but the value at the critical velocity is not a constant, as usually assumed. For tubes of about fifty diameters in length, both equations would give the same loss of head.

4.—In calculating the discharge through orifices, the difficulty is that the coefficient, c, in the Torricelli formula is not a constant, but varies with the diameter, head, and viscosity, especially for small diameters and low heads such as might be used in viscosimeters and carburetors.

5.—The Torricelli formula must be abandoned for short tubes, because a table of coefficients based on four variables—head, diameter, viscosity, and length—becomes too complicated. Recourse, therefore, must be had to Equation (1) or Equation (6), care being taken not to use tubes so short that the discharge would be the same as for an orifice, and these or other equations containing the length of tube would not apply. The Couette and kinetic energy corrections cannot be neglected with short tubes.

* P. Ronceray, *Annales de Chimie et de Physique*, Vol. XXII (1911), p. 107.

DISCUSSION

ARTHUR C. PRESTON, ESQ.* (by letter).—That part of the author's paper which deals with flow in long pipes will be of great interest to engineers who have to design piping systems for liquids of unknown flow characteristics; that is, liquids, usually other than water, the friction losses of which are not known from previous observation or experiment. The author performs a service in outlining means for computing friction loss based on the fundamental fluid characteristics, viscosity and density. The methods offered are:

(*a*).—The use of Rayleigh's formula, Equation (4), with the curve for values of

$$F\left(\frac{v\,d\,\rho}{\mu}\right)$$

as developed by Stanton and Pannell†; this equation also may be found developed by Buckingham.‡

(*b*).—The use of Herschel's formulas Equations (1) and (6), or their modified forms, Equations (7) and (8).‖

Since the latter two equations may be derived from Rayleigh's formula, the method to be chosen will be that which appeals to the investigator as most convenient. Equation (4*a*) is general, applying both to viscous and to turbulent flow, but it has the variable factor

$$F\left(\frac{v\,d\,\rho}{\mu}\right)$$

which cannot be expressed by one simple equation because it has two main branches, one giving the value of the function for viscous flow (when less than 2 000) and one giving the value for stable turbulent flow (when greater than about 2 900), as shown by the curve previously referred to. The indefinite branch between the values of 2 000 and 2 900 for the criterion may also be given an approximate location on the curve, and the use of this method is then quite simple. Having computed the value of the criterion for the case in hand, we can take the value of $\frac{R}{\rho\,v^2}$ from the curve. As Mr. Herschel shows that

$$R = \frac{h}{l} \cdot \frac{\rho\,g\,d}{4}$$

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* Cordoba, Spain.

† "Collected Researches", National Physical Laboratory, p. 302, Fig. 3, Vol. XI (1914); Fig. 4, Vol. XIII (1916).

‡ "Model Experiments and the Forms of Empirical Equations", *Journal*, Am. Soc. of Mech. Engrs., June, 1915.

‖ Mr. Preston has called attention to errors in the original form of the author's Equations (4), (7), and (8), and the latter has authorized their correction as follows:

$$R = \rho\,v^2\,F\left(\frac{v\,d\,\rho}{\mu}\right) \qquad (4)$$

If $\gamma = \rho$, this reduces to

$$\frac{h}{l} = \frac{4\,v^2}{g\,d}\,F\left(\frac{v\,d\,\rho}{\mu}\right) \qquad (4a)$$

$$\frac{h}{l} = \frac{C\,\mu\,v}{\gamma\,g\,d^2} \qquad (7)$$

$$\frac{h}{l} + \frac{0.1580\,v^{1.75}}{\gamma\,g\,d^{1.25}}\left(\frac{\mu}{\rho}\right)^{0.25}\rho \qquad (8)$$

(noting that $\gamma = \rho$ when friction loss is measured in vertical head of the liquid itself), the computation of h is a simple matter of substitution in Equation (4*a*). Values of $\frac{\mu}{\rho}$, if not otherwise known, may be determined for liquids more viscous than water by the use of a standardized commercial viscosimeter as described in Mr. Herschel's former work.* When the equations and tables as given in that paper are used, all computations must be in c. g. s. units.

The use of Equations (7) and (8) avoids the variable factor, but the equations are not general. The first applies only to viscous flow, and the second only to stable turbulent flow, so that, in order to know which to use, it is necessary to make a preliminary investigation of the value of the criterion. Computations in the intermediate stage between viscous and stable turbulent flow seem to be left out of account in this method, although the author calls attention to the importance of this régime.

It may be concluded that the two methods require about equal amounts of computation; (*b*) may be preferred for its more rigorously mathematical forms, while (*a*) is more general and exhibits better the complex situation at the transition from viscous to turbulent flow.

The author states that for a given liquid at constant temperature, Equation (6) reduces to the Chezy formula, $v = c\sqrt{rs}$. It is interesting to note that from this equation, or from its progenitor, Equation (4), may also be derived another well-known hydraulic formula,

$$h = F \frac{l}{d} \frac{v^2}{2g} \qquad (17)$$

Equation (4) is:

$$R = \rho v^2 F \left(\frac{v d \rho}{\mu} \right)$$

If we designate the "function of $\frac{v d \rho}{\mu}$" by the letter, f, and substitute for R its value

$$\frac{h}{l} \cdot \frac{\rho g d}{4}$$

this becomes at once

$$h = 8 f \frac{l}{d} \frac{v^2}{2g} \qquad (18)$$

which evidently is the same as Equation (17) with an f one-eighth as large.

This formula is one which in spite of its familiarity is sometimes misinterpreted by engineers who have occasion to use it. It is often supposed that because the exponent of v is 2, friction loss for a given pipe and liquid increases as the square of the velocity, the fact being overlooked that f is a variable coefficient which itself varies with the velocity. This erroneous statement is seen sometimes in print. When it is realized that, as shown by Mr. Herschel, h varies approximately as the 1.75th power, there is a tendency to discredit the formula altogether, on the assumption that it is entirely irrational and requires the use of an arbitrarily varying coefficient to drag the computed result back into line with

* *Technologic Paper No. 112*, 1919.

fact. To any reader who may have failed to find a thorough discussion of this formula in the hydraulic textbooks which quote it, the foregoing derivation may be of interest as showing that it has, after all, a rational and satisfactory basis.

From the curves of Stanton and Pannell the author's Equations (7) and (8) may be derived in a very easy and convenient manner. These curves show values of $\frac{R}{\rho v^2} = \frac{f}{8}$ as ordinates, and values of $\frac{v d \rho}{\mu}$ as abscissas. The curves show the two principal branches which have been spoken of, one for viscous and another for stable turbulent flow. The equation of each of these branches may be determined by logarithmic plotting, which gives values of f in terms of $\frac{v d \rho}{\mu}$ for each of the principal régimes of flow. The resulting equation for viscous flow is:

$$f = \frac{64 \mu}{v d \rho} \qquad (19)$$

and this being substituted in Equation (17), gives,

$$\frac{h}{l} = \frac{32 \mu v}{\rho d^2 g}$$

which is the author's Equation (7) or Equation (1), a modified form of Poiseuille's equation. It will be noted that when the head is measured in vertical units of the liquid itself, $\gamma = \rho$, as previously stated.

The equation for turbulent flow is

$$f = 0.316 \left(\frac{\mu}{v \rho d} \right)^{0.25} \qquad (20)$$

and this being substituted in Equation (17), gives,

$$\frac{h}{l} = \frac{0.158 \, v^{1.75} \left(\frac{\mu}{\rho} \right)^{0.25}}{g d^{1.25}}$$

thus corroborating Mr. Herschel's Equations (6) and (8) when $\gamma = \rho$ as before.

An important corollary to be drawn from this work may be stated as follows: All liquids having equal viscosity-seconds as measured by an ordinary commercial viscosimeter have equal friction heads when moving at the same rate in the same pipe, whether in viscous or in turbulent flow. It may readily be seen from Equations (7) and (8) that liquids of equal kinematic viscosity, $\frac{\mu}{\rho}$, have equal values of h under the circumstances stated; and it is shown in *Technologic Paper No. 112* that liquids of equal kinematic viscosity have equal commercial viscosity-seconds (Saybolt, Redwood, or Engler), whence the proposition as stated.

This is a fact which if thoroughly understood would smooth the way for many perplexed designers confronting an unfamiliar problem. It is possible to make tables of friction head lost in flow for varying kinematic or commercial viscosities, rates of discharge, and sizes of pipe. Such tables may be conveniently based on one of the three commercial instruments mentioned; the friction heads either may be observed in direct experiment, or, for smooth drawn pipes such as were used by Stanton and Pannell, they may be computed by the methods outlined

by the author, using the equations and tables of *Technologic Paper No. 112* to make the necessary conversion of commercial viscosity-seconds into physical units. The results may be applied without very great error to ordinary wrought-iron and cast-iron pipes.

WINSLOW H. HERSCHEL,* ESQ. (by letter).—The writer would call attention to the fact that R. S. Danforth, Assoc. M. Am. Soc. C. E., has published a pamphlet entitled "Oil Flow in Pipe Lines", 1921, in which he has given diagrams of pressure lost in flow for various kinematic and Saybolt Universal viscosities, Baumé gravities, rates of discharge, and sizes of pipe, as suggested in Mr. Preston's last paragraph.

Attention is also called to a paper by E. Parry, "On a Theory of Fluid Friction and Its Application to Hydraulics",† which contains tables of values of log $\frac{v\,d\,\rho}{\mu}$ for water, for various temperatures, and values of $v\,d$.

* Associate Physicist, U. S. Bureau of Standards, Washington, D. C.
† *The English Electrical Journal*, Vol. I (1920), p. 146.

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Paper No. 1476

A METHOD FOR ADAPTING THE RECORDS OF STREAM FLOW AT ONE POINT TO ANOTHER POINT ON THE SAME STREAM

By H. W. Dennis,* M. Am. Soc. C. E.

With Discussion by Messrs. Charles P. Dunn, W. L. Huber, and H. W. Dennis.

Synopsis.

In this paper the writer will undertake to show the results of a rather extended study of the flow of the Kern River in the State of California, wherein it was desired to make use of a long period of observations of stream flow taken on this river at a point where the water is diverted for irrigation purposes, and to apply these records to a point about 75 miles distant on the same river where it was contemplated to divert the water for power purposes. Before the power development could proceed, and, in fact, before its desirability could be established definitely, it was necessary to decide the probable performance of the plant, not only for the proper design of the structures, but for the determination of the probable output in relation to the anticipated cost. This is the same problem always encountered when any development of magnitude is contemplated, either in hydro-electric power or in irrigation, for in any such anticipated work the two things which must be predicted are: First, the cost of the work, and, second, its performance.

The writer will undertake to show that the Kern River, which is a typical Southern California stream, has entirely different characteristics during different periods of the year, and that the direct comparison of stream flow between two points must take into consideration the variation in stream characteristics throughout the year, or else the results obtained will lead to erroneous conclusions. He will undertake to show that without consideration of these variable characteristics the resulting computations of low-water run-off, during the period when the river is not supported by storm, will be too high, and that the conclusions as to run-off during the period when the stream is supported by the water from melting snow will be too small. If these conclusions are substantiated, he will show that the results from the application of ordinary methods will be misleading, not only as

* Los Angeles, Cal.

to the question of the cost of the work and the design of the structures, but as to the performance on completion.

Data Used and Assumptions Made.

The Kern River, which is the subject of this study, is a typical Southern California stream in the following respects: It originates in the high altitudes where the precipitation during the winter months is principally retained in the form of snow. Its drainage area is made up largely of mountainous country wherein the natural rock is provided with earth covering to a comparatively small depth, and for many large areas there is no cover whatever. The slope of its river bed is fairly uniform from the mountains to the head of the débris cone, below which point the slope is materially less, the stream meanders to some extent, and, comparatively close to the mouth of the canyon, the water is diverted for irrigation purposes.

The first diversion of any large quantity of water on the Kern River occurs at a point a few miles below the mouth of the canyon, and numerous canals of great capacity are led away from the river until, during periods of comparatively low flow, the entire discharge of the stream has been diverted. The waters of the river were appropriated for irrigation purposes, many years ago, under the California laws. An adjudication of the rights of the various owners was made in 1888. In 1894 there was established, a short distance above the first of these large diversions, a gauging station known as the "First Point of Measurement," and at this point records of the daily flow of the Kern River have been kept continuously since that date with the exception of approximately eight months, namely, from July 1st, 1907, to March 1st, 1908. These records at the "First Point of Measurement" have been kept with great accuracy, in spite of the fact that the stream bed has a bottom of shifting sand. The writer is indebted to the engineers of the Kern County Land Company and of the Water Resources Branch of the United States Geological Survey, for a great amount of data in connection with these measurements.

About 75 miles above the "First Point of Measurement", the Southern California Edison Company, a corporation engaged in the development and sale of hydro-electric power, contemplated, and is now nearing the completion of, a hydro-electric generating station. There was little data as to the flow of the Kern River at the point of proposed diversion, except an occasional measurement of the stream flow. Gaugings were started at this point at the beginning of 1912 and have been taken continuously since that date. Initial comparisons of the flow of the river at the two points were made, and as further data were secured the method herein described was developed.

Between the point of proposed diversion on the north fork of the Kern River, and the station known as "First Point of Measurement," the north fork and the south fork of the river join to make the main stream. It is a matter of observation that during the period of comparatively low flow in the river very little water is contributed by the south fork. It is the writer's opinion that this fact is explained by the topographical conditions of the water-shed and by the irrigation uses in the South Fork Valley. There are other streams and stream beds entering the river at various intervals, but it is a further matter of observation that these

streams are small individually, and that during the period of low flow in the main river most of these small tributaries are entirely dry. The conclusion seems to be justified, and it is substantiated by the data to be shown, that their influence is, in general, fully taken into consideration in the method to be described.

The records taken at the point of proposed diversion have been published in the Water Supply Papers of the U. S. Geological Survey, under the title "Kern River near Kernville, California", and the location is described as being at the base of Fairview Mountain. In this paper this station has been called Fairview.

In determining the relation of flow between the "First Point of Measurement" and Fairview, the fundamental assumption has been made that there is a lag of twenty-four hours between the two points. That is to say, the assumption has been made that the water passing Fairview will reach the "First Point of Measurement" twenty-four hours later. It is realized that this assumption is not strictly in accord with facts; that the velocity of the water is less during the period of low flow than during the period of high flow, and that the comparative flow with a rising stream differs from the corresponding flow with a falling stream. The condition of the record, however, by which the stream flow has been computed at each point for each calendar day, does not permit the taking into consideration of these known facts, and it has been felt that the allowance of the lag of twenty-four hours will best serve the case in hand.

It is further realized that the stream is subject to storms which may be local in character, which storms would show a rather large run-off at Fairview, but that the peak of such discharge would be much less pronounced on reaching "First Point of Measurement." In making these comparisons, therefore, the discharges for each day have been combined into groups of four, five, or six observations, by dividing each month, in general, into six separate groups. Each point is assumed, therefore, to have practically the same weight as any other point. Further, in assembling these observations, the five highest observations at "First Point of Measurement" during any one month have been combined with the observations of the corresponding days at Fairview, and, similarly, for each other group five or six observations have been taken in numerical sequence rather than an average of the observations of five consecutive days of the month, irrespective of the conditions of flow during those five days.

The comparison here presented covers the entire period of simultaneous records from 1912 to 1919, inclusive, except for a part of 1919, when there was known to have been an error in the Fairview records. Figs. 1 to 15, inclusive, have been prepared to show the relation of the flow of the Kern River comparing "First Point of Measurement" with Fairview.

Ordinary Method of Making Comparison of Flow.

It is the writer's opinion that the ordinary method of making a comparison of the flow of water at two stations on the same river would involve plotting simultaneous observations, using the flow of the river at one station as ordinates and the flow at the other station as abscissas. The result of applying this method in the present instance is shown by Figs. 1, 2, 3, and 4. It will be noted that the separate observations form a diagram in which there is little fluctuation from a line at the point of comparatively low flow, and large fluctuation from that line as greater discharge is considered. Fig. 1 shows the relation up to 4 000 sec-ft. at "First

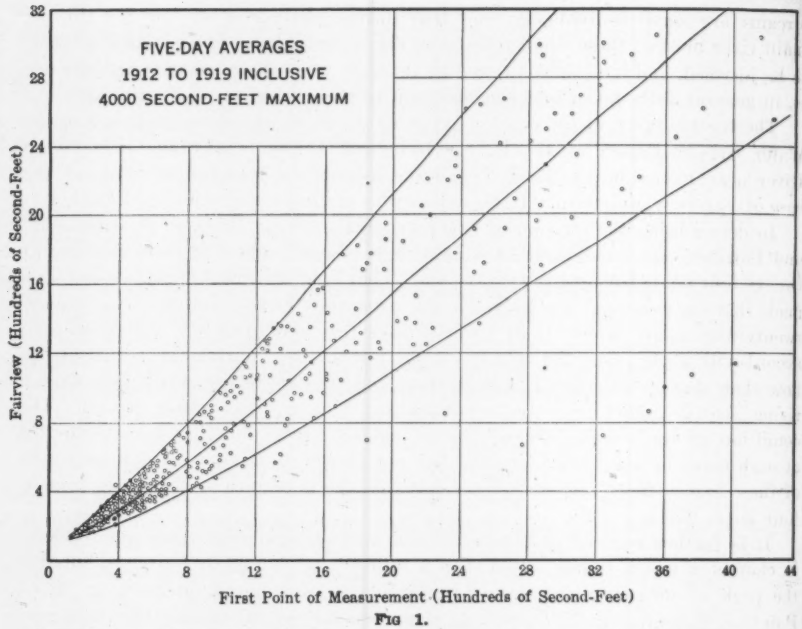
FIVE-DAY AVERAGES
1912 TO 1919 INCLUSIVE
4000 SECOND-FEET MAXIMUM
Fairview (Hundreds of Second-Feet)
First Point of Measurement (Hundreds of Second-Feet)
0
4
8
12
16
20
24
28
32
36
40
44

FIG 1.

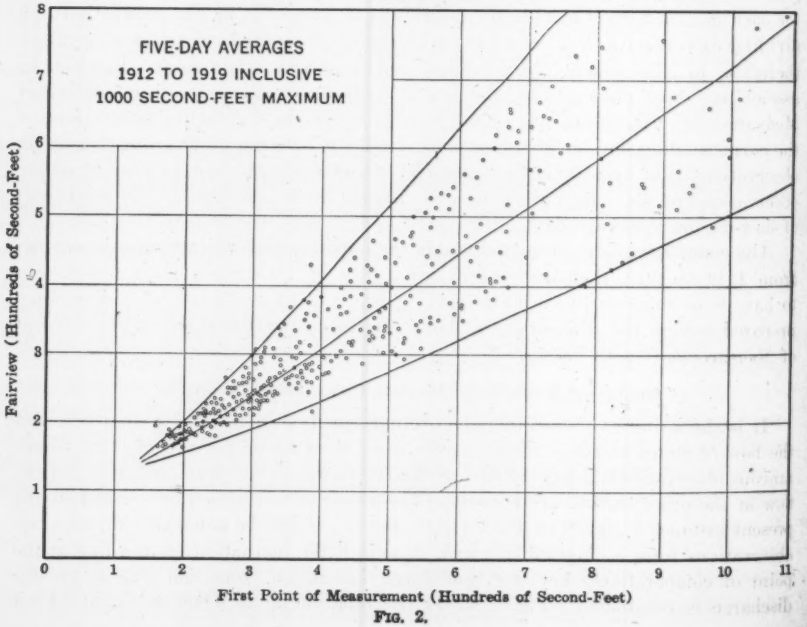
FIVE-DAY AVERAGES
1912 TO 1919 INCLUSIVE
1000 SECOND-FEET MAXIMUM
Fairview (Hundreds of Second-Feet)
First Point of Measurement (Hundreds of Second-Feet)
0
1
2
3
4
5
6
7
8
9
10
11

FIG. 2.

Point of Measurement." Fig. 2 shows the same data on a larger scale up to 1 000 sec-ft. Fig. 3 shows the same relation using monthly averages, and Fig.

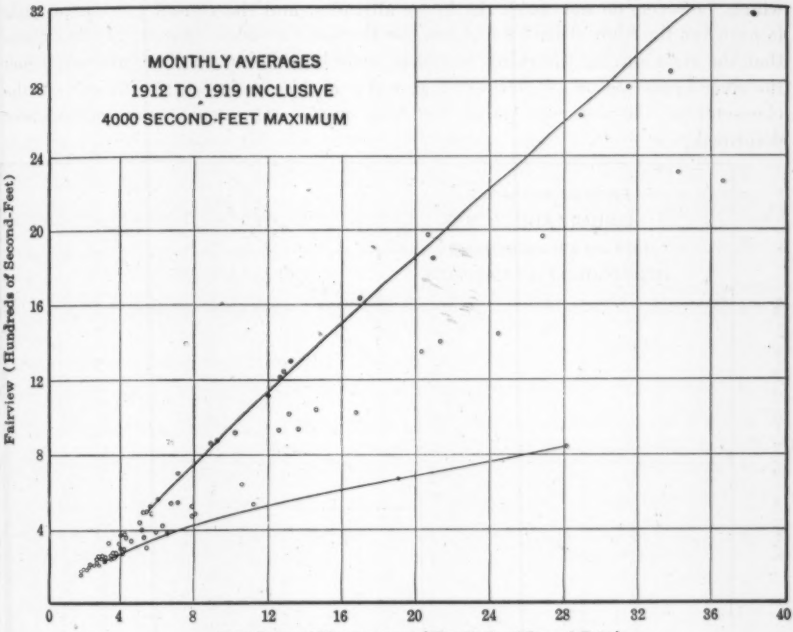

FIG. 3.

4 shows the annual averages. A reasonable curve may be drawn on Fig. 4, but a single curve on each of the other diagrams will show wide variation from actual observations.

CHANGE IN CHARACTERISTICS OF THE WATER-SHED.

In common with other Southern California streams the head-waters of which are in the Sierras, the annual precipitation on the water-shed of the Kern River is much higher in the high altitudes than in the low altitudes. Furthermore, such precipitation is distributed differently and reaches the stream bed differently in the high altitudes as compared with the low altitudes. During the period of spring rains temperatures are low, resulting in snow on the mountains with low run-off, and rain in the low altitudes with comparatively high run-off. This condition means that the run-off per square mile at this season of the year is low for the high altitudes and high for the low altitudes.

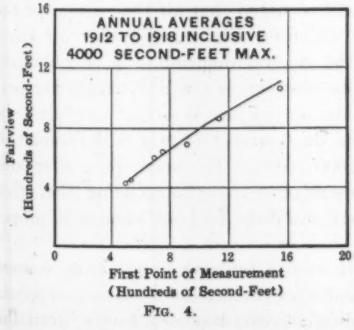

FIG. 4.

During the months of the early summer, namely, May, June, and July, the accumulation of snow in the high altitudes is supplemented by occasional storms which, however, do not reach the lower altitudes, and the run-off per square mile is high for the high altitudes and low for the low altitudes. From this, it follows that the river during the rainy season is entirely different in characteristics from the river in the season of melting snow, and it is this fundamental difference in the character of the water-shed that has been used in developing the method here described.

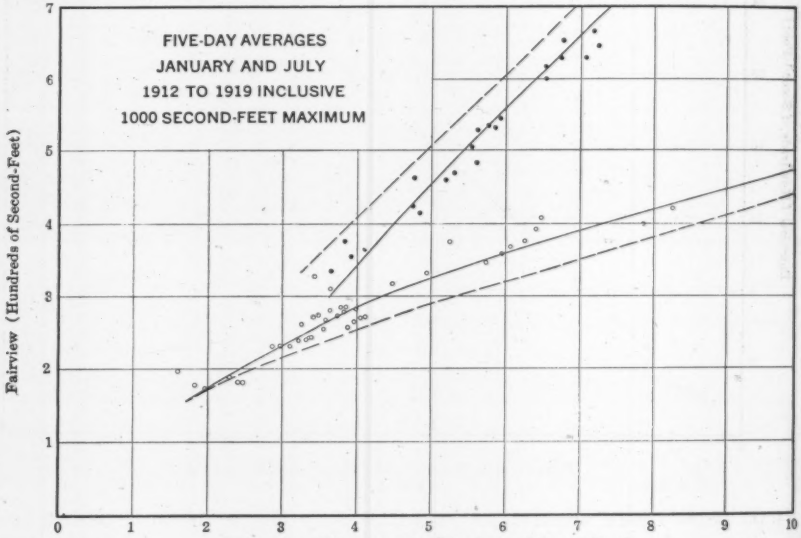

FIG. 5.

DESCRIPTION OF METHOD.

Fig. 5 shows the comparison of the discharges in the month of January; also the comparison for the month of July. It will be noted that all the points for the January flow are along the bottom of the diagram, Fig. 1, and that all those for the July comparison are at the top of the general diagram, Fig. 1. On this diagram the solid line through each set of observations is the line which will best fit all the observations for that month, and the dotted line is a line which will be explained hereafter. Fig. 6 shows the same data spread over a wider range of comparison, namely, with a maximum of 4 000 sec-ft. The succeeding diagrams will show that the slope of the lines representing comparison during the other months will lie between the curves for January and July, and will change in proper order.

It will be noted on Fig. 7, which shows the record for each month from August to February, inclusive, that whereas the general slope of the line for August approximates 45°, the general line for the succeeding months becomes flatter, until the slope for January is found to be, as already mentioned, along the bottom of the

observations shown on Fig. 1. It is also to be noted that the observations plotted on Fig. 7 include all the observations for all these separate months, except the high flows of January and February. The full line through each set of points is the line which appears best to fit those points, whereas the dotted line is the line to be explained hereafter.

On Figs. 8, 9, 10, and 11, the points are shown for the succeeding months to complete the annual cycle. It is here to be noted that the gradual change in the slope of the line is from the flat slope of January to the slope of nearly 45° for July. On Figs. 10 and 11 the irregular lines joining various groups of points connect observations of separate years. The month of May, particularly, as shown on Fig. 10, seems to be more erratic than any of the others. It will be noted

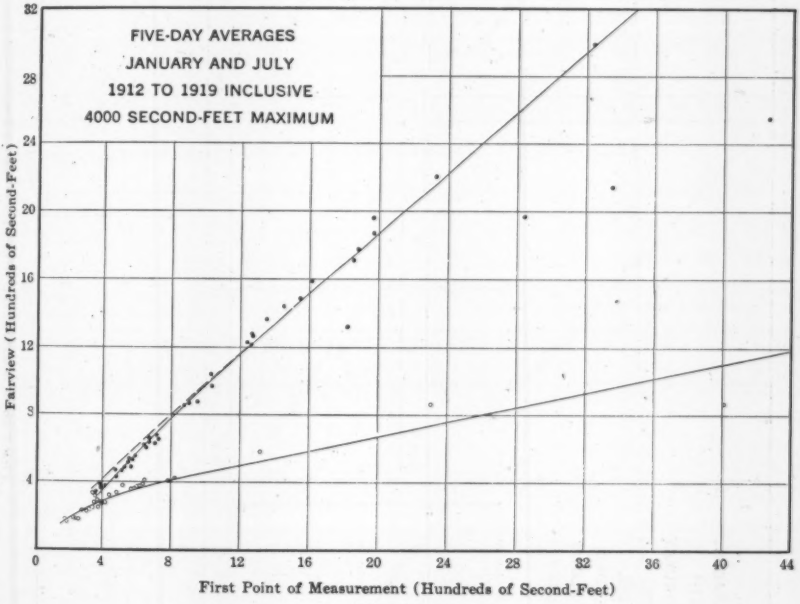

FIG. 6.

that the general slope for each year is steeper than the line which has been drawn through the entire combination, and it may be suggested that possibly this is due to the fact that the month of May may be expected to provide heavy local storms throughout the water-shed, as well as the beginning of the general run-off due to the melting snow in the high mountains.

On Figs. 5 to 11, inclusive, all the points have been shown which appeared on Fig. 1, that is, all the observations for the entire period of record up to a maximum of 4 000 sec-ft. have been plotted separately by months, and it has been shown that the fundamental assumption of change in characteristics of the water-shed is truly borne out by separately plotting the relation of the stream flow month by month.

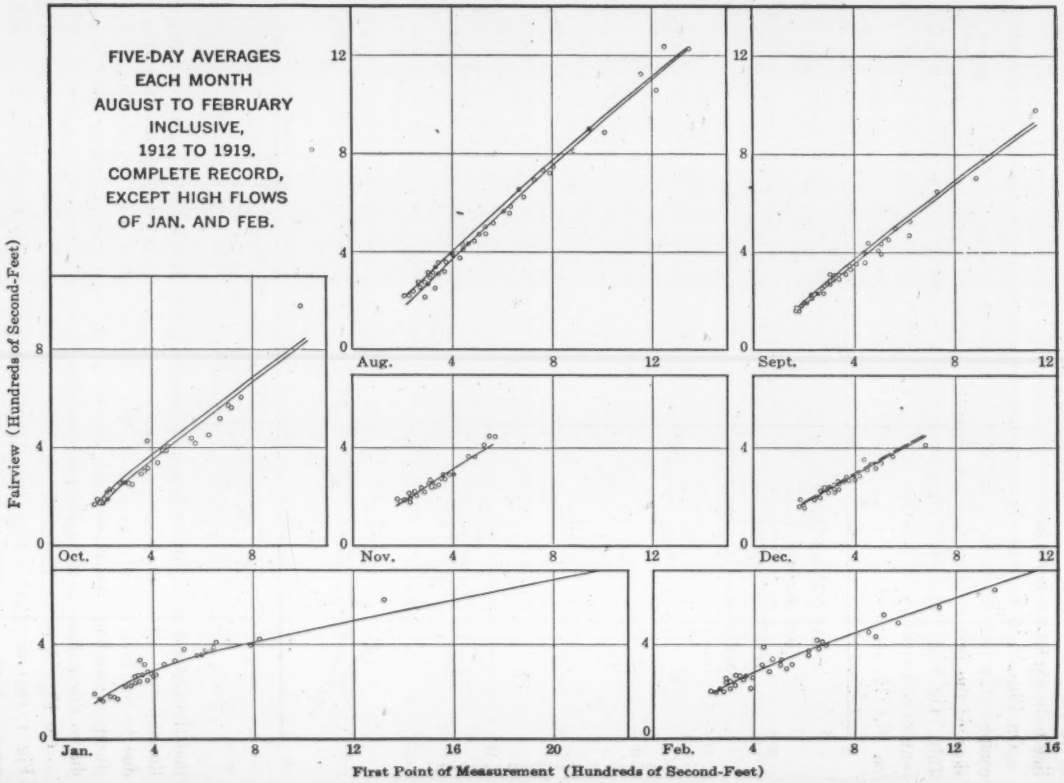
FIVE-DAY AVERAGES
EACH MONTH
AUGUST TO FEBRUARY
INCLUSIVE,
1912 TO 1919.
COMPLETE RECORD,
EXCEPT HIGH FLOWS
OF JAN. AND FEB.
Aug.
Sept.
Oct.
Nov.
Dec.
Jan.
Feb.
Fairview (Hundreds of Second-Feet)
First Point of Measurement (Hundreds of Second-Feet)

FIG. 7.

On Fig. 12 is shown the assembly of the various monthly curves. This diagram shows the curves as actually determined by the points on each separate month, and Fig. 13 shows the same data enlarged for the lower flows of the river, that is, for those discharges which at "First Point of Measurement" are less than 1 000 sec-ft.

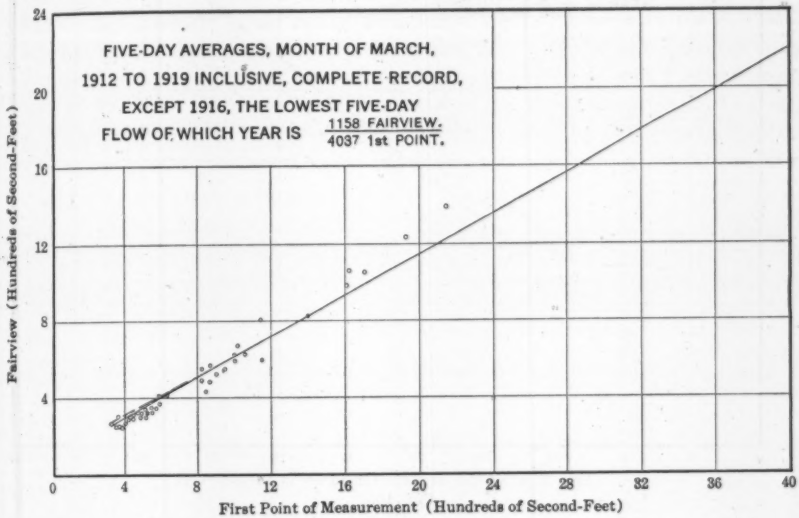

FIG. 8.

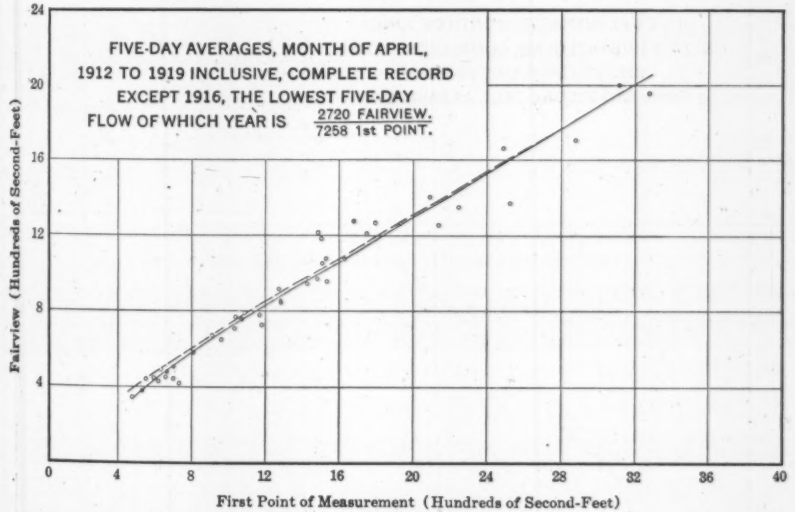

FIG. 9.

It is obvious, of course, that the division of these records into monthly periods and the preparation of separate curves for each month has been a matter of convenience only. The river does not know January from February, and it seems

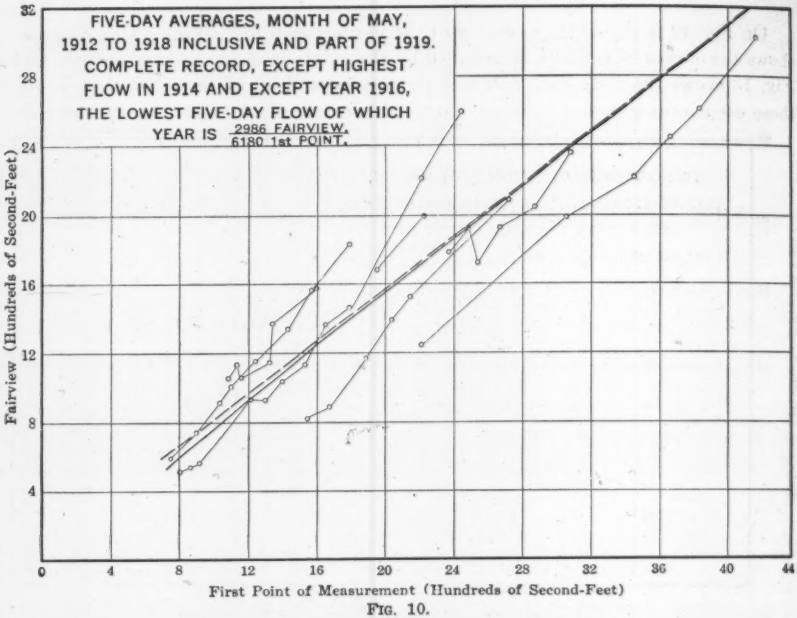
FIVE-DAY AVERAGES, MONTH OF MAY,
1912 TO 1918 INCLUSIVE AND PART OF 1919.
COMPLETE RECORD, EXCEPT HIGHEST
FLOW IN 1914 AND EXCEPT YEAR 1916,
THE LOWEST FIVE-DAY FLOW OF WHICH
YEAR IS 2986 FAIRVIEW.
6180 1st POINT.
Fairview (Hundreds of Second-Feet)
First Point of Measurement (Hundreds of Second-Feet)

FIG. 10.

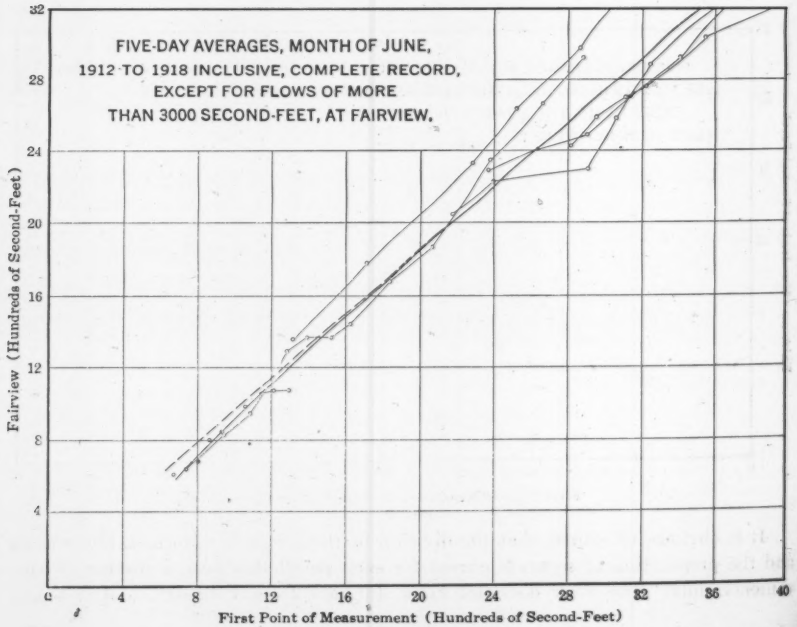
FIVE-DAY AVERAGES, MONTH OF JUNE,
1912 TO 1918 INCLUSIVE, COMPLETE RECORD,
EXCEPT FOR FLOWS OF MORE
THAN 3000 SECOND-FEET, AT FAIRVIEW.
Fairview (Hundreds of Second-Feet)
First Point of Measurement (Hundreds of Second-Feet)

FIG. 11.

proper that all the curves should be similar in shape. It will be noted on these diagrams, and particularly on Fig. 13, that the curve for March crosses the curves for January and February; that the curve for May would apparently, if produced, cross the curve for April, and that the curve for June is quite different in direction from the curve for July. If it is true that the characteristics of the watershed change gradually from month to month, then it follows that the monthly curves should be similar in shape, and the variations to which reference has just been made would not occur if more observations were extended over a longer time. Accordingly, Figs. 14 and 15 have been prepared, on which are shown the curves by months adjusted to conform to the assumption that the curves should move

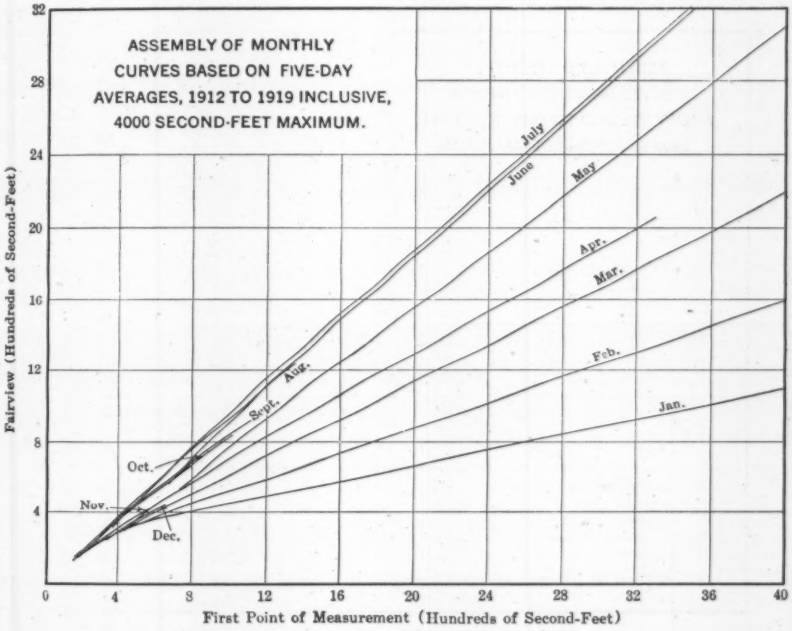

FIG. 12.

gradually from one general slope to another. Fig. 14 shows these discharges to a maximum of 4 000 sec-ft. and Fig. 15 shows the larger scale of the lower part of the curve, that is, up to a maximum of 1 000 sec-ft. It will be noted particularly that the curves change in definite order in accordance with the sequence of the months themselves.

The dotted lines shown on the preceding diagrams are the adjusted monthly curves as plotted on Figs. 14 and 15.

APPLICATION OF THE VARIOUS METHODS.

In order to show the practical application of the method of using monthly curves in comparison with a single curve through the general diagram, Fig. 1, Fig. 16

has been prepared, which shows the hydrograph of the flow of Kern River at Fairview for 1917. This diagram shows: First, the actual flow of the river; second, the flow as computed from the unadjusted monthly curves (Curve *A*); third, the flow as computed from the adjusted monthly curves (Curve *B*); and, fourth, the flow as computed from the single curve which was drawn through the general diagram (Curve *C*).

The computed curves are based on the actual observed flow of the river at "First Point of Measurement." Certain lines are also shown on Fig. 16, indicating the percentage of error by which the computed flow varies from the actual flow. The hydrographs are based on averages of five consecutive days, dividing each

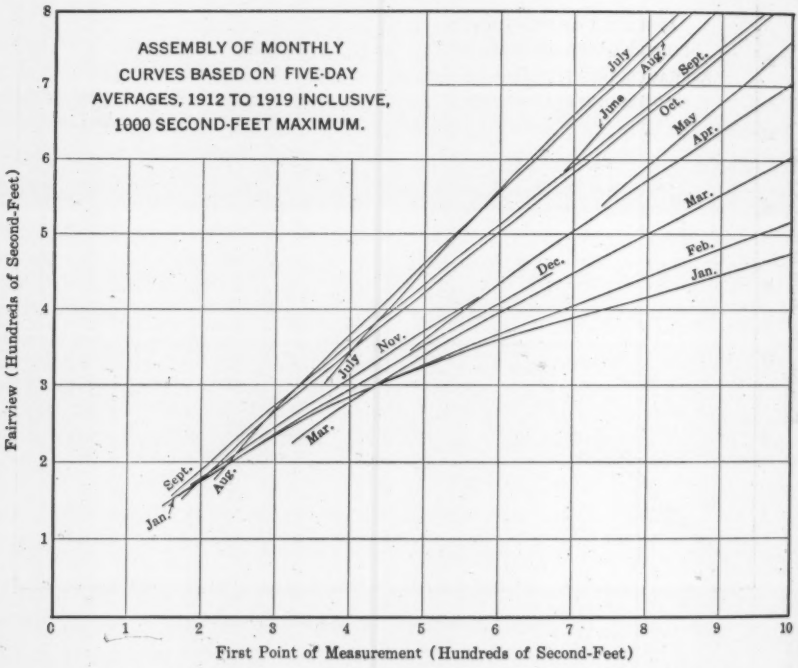

FIG. 13.

month into six periods, and the curves showing percentages of error take the average of each curve for each month. Table 1 shows the percentage of error for each month and for the total of the year for each of the three computed curves.

It will be noted that the maximum error based on the entire discharge for any one month from the unadjusted monthly curves is 7.9% which occurred in December, and that there are only three other months during which the error reaches approximately 5 per cent. It will be noted, further, that the greatest error of any one month under the adjusted monthly curves is 11.5%, which occurs for December, that there are two other months where the error is 10.4%, and that the errors for the other months are less than 7.5 per cent.

TABLE 1.—PERCENTAGE OF ERROR BY MONTHS, COMPARING COMPUTED FLOW WITH ACTUAL FLOW (1917).

Month.	CURVE A.		CURVE B.		CURVE C.	
	+	−	+	−	+	−
January	.....	0.006	.....	0.104	0.280	.....
February	.....	.....	.....	0.104	0.247	.....
March	0.003	.....	0.013	.....	0.181	.....
April	.....	0.009	0.006	.....	0.159	.....
May	0.030	.....	0.072	.....	0.072	.....
June	0.051	.....	0.085	.....	.....	0.092
July	.....	0.010	0.005	.....	.....	0.221
August	0.022	.....	0.064	.....	.....	0.177
September	.....	0.048	.....	0.033	.....	0.173
October	.....	0.012	0.016	.....	.....	0.090
November	0.042	.....	0.065	.....	0.017	.....
December	0.079	.....	0.115	.....	0.091	.....
Total	+ 0.018		+ 0.028		+ 0.003	

Considering the application of the single curve for the entire year, it will be noted that there is only one month where the error is less than 5%; that there are only four other months where the error is less than 10%; and that for the remaining seven months the error is in excess of 15 per cent. It will be noted particularly that the results for the single curve over the entire year substantiate

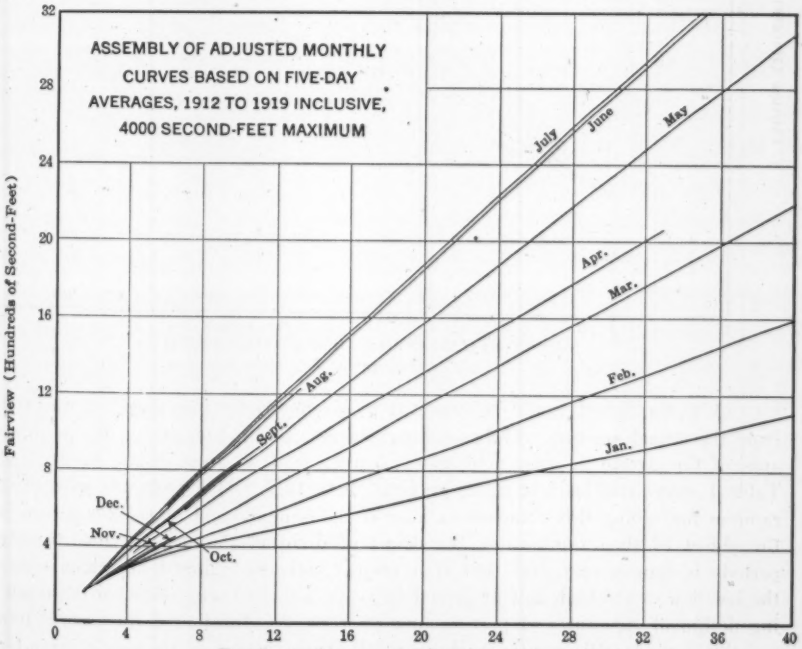

FIG. 14.

the conclusion that the use of such a curve gives results that are too high during conditions of low flow and too low under conditions of high flow where the river is supported by melting snow.

Considering the total for the entire year, it will be observed that any of the three methods gives for this particular year a result for the total run-off which is less than 2.5% in error, and that the single curve for the entire year gives a result which is less than 0.5% in error.

The records at both these stations are based on current-meter observations, for which the error in any individual observation may be as much as 5 per cent. It is probable that these errors would be compensating, so that the total discharge

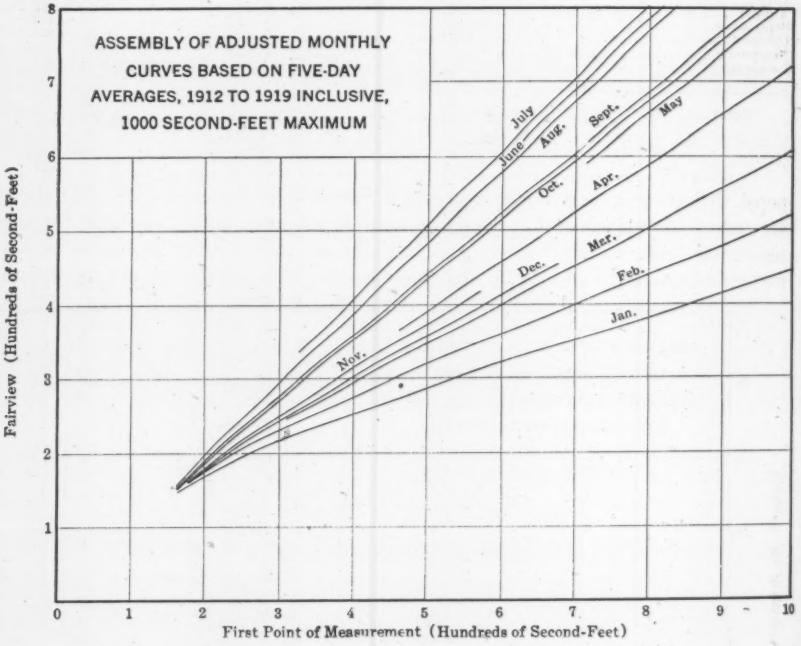

FIG. 15.

for any given period would be expected to be materially less than 5% in error from the actual amount. The use of the old records in determining the performance of the stream in years gone by necessarily leads to approximate results, and Table 1 shows conclusively in its application to 1917, which year was selected at random for taking this comparison, that if the annual yield of the water-shed is the object of the examination, the detail of dividing the records into monthly periods is unnecessary, but that if a project is being planned on which either the low flow or the high flow of preceding years is to have any effect on the resulting design of structures or performance of work, the division of the records into monthly periods will give much closer results than the use of the single curve for the entire year.

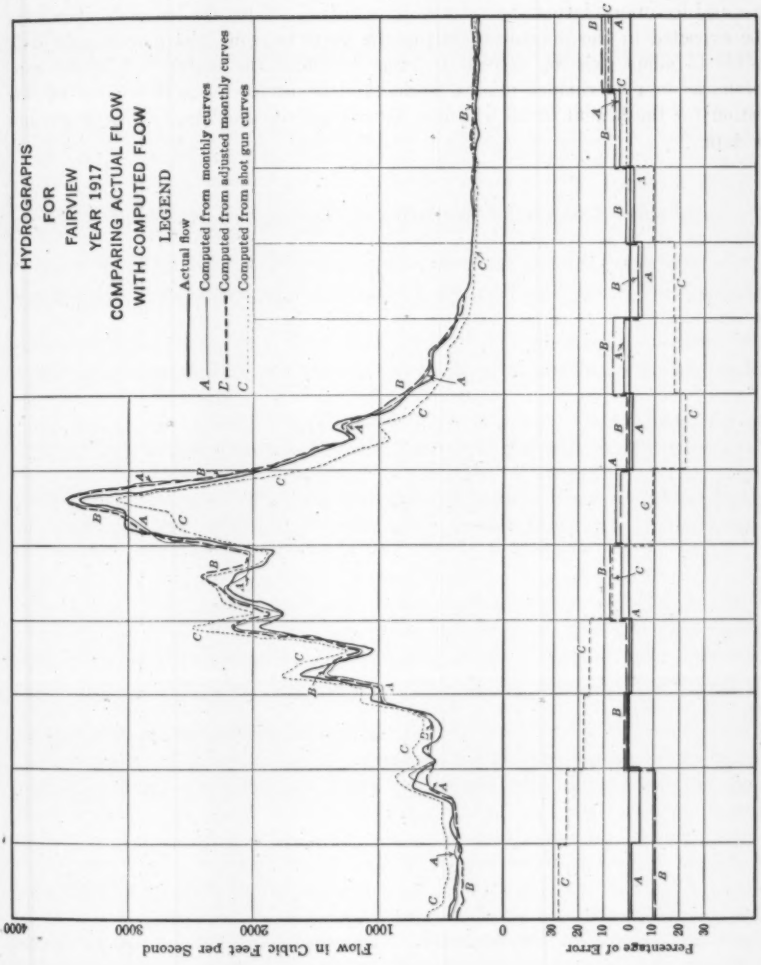
HYDROGRAPHS
FOR
FAIRVIEW
YEAR 1917
COMPARING ACTUAL FLOW
WITH COMPUTED FLOW
LEGEND
Actual flow
A ——— Computed from monthly curves
B - - - - - Computed from adjusted monthly curves
C ..... Computed from shot gun curves
Flow in Cubic Feet per Second
4000
3000
2000
1000
0
Percentage of Error
30
20
10
0
10
20
30
Jan.
Feb.
Mar.
Apr
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.

FIG. 16.

The writer feels that the great amount of detail work which has been necessary to develop all the monthly curves which have been shown would be unnecessary in the practical application of this problem to another stream which has the characteristics of the Kern River, and believes that reasonable approximations of the flow by months might be obtained by constructing the January curve and the July curve and by interpolating the intermediate curves. With the great developments to be expected in the Southwest during the years to come, the more nearly it is possible to adapt existing records to other locations the better will be the conclusions as to performance of any works, and he invites open discussion of this question for the benefit of all who may be engaged in problems involving stream-flow data.

DISCUSSION

CHARLES P. DUNN,* ASSOC. M. AM. SOC. C. E. (by letter).—The problem which Mr. Dennis analyzes so skilfully, is one of rapidly increasing importance. As complete utilization of the flow of streams is approached, every possible effort must be made to avoid guesswork in the determination of the quantity of water available.

Present-day machinery is so highly efficient that very little advancement in the way of mechanical, hydraulic, and electrical efficiency can be expected, but the dividend-paying capacity (which is the real measure of efficiency) of power developments and irrigation systems can be increased materially by making careful studies of stream flow such as are indicated by Mr. Dennis and also by L. Standish Hall, Assoc. M. Am. Soc. C. E., in his excellent paper on "The Probable Variations in Yearly Run-off as Determined from a Study of California Streams".†

If a power installation is designed without due appreciation of the importance of using every bit of evidence that can be obtained as to the quantity of water available, one may either have too large a plant, with a part of the machinery practically idle, or too small a plant for the stream, which may necessitate an additional installation, at a later date, at a much higher unit cost than would have been necessary if all the work had been done at one time. Either condition causes a waste of money.

If the dividend-paying capacity of a plant as a whole is considered, a small error in determining the quantity of water available will utterly wipe out the results of years of painstaking effort on the part of designers of machinery to obtain high efficiencies. Machine efficiency is a definite thing which can be easily measured; financial efficiency involves much more complicated problems, and is not so easily measured; a technically trained man is likely to give too much attention to the former, and too little to the latter.

In fixing the size of a development, engineers are often confronted with such erratic variations and apparent discrepancies in the recorded stream flow that they are tempted to choose, by an inspection of the records, a figure which looks about right, because it is all a gamble with the "weather man" anyway. The designer must make a wager with the "weather man," but if he places his wager scientifically, he has a better chance to win.

The Columbia River, one of the greatest undeveloped powers in the world, is a good illustration of the usefulness of the method outlined by Mr. Dennis. Records of flow at The Dalles, Oregon, have been kept for forty-two years. A comparative study will make these records available for use in the design of developments at a number of sites on the Upper Columbia where records have been kept for a much shorter period.

It is to be hoped that in the near future extensive snow-survey service will be carried on by the Federal Government. Such records extending over a period sufficiently long to establish a correct relation between snowfall and run-off, will have a value many times their cost in aiding the regulation of reservoirs for power and irrigation. A foreknowledge of the season's run-off in many cases will save large fuel bills, and will reduce the size of the steam auxiliaries necessary.

* Seattle, Wash.

† See p. 191.

W. L. HUBER,* M. AM. SOC. C. E. (by letter).—Those who have had occasion to study the stream flow from rather limited areas of the upper portions of the water-sheds of streams draining the Sierra Nevada Mountains and, particularly, of those streams draining the western slope of that range, should readily acknowledge their indebtedness to Mr. Dennis for his thorough study of the relation between the flow from one of these limited high areas and that from practically the entire water-shed of the Kern River. The study presented in the paper is the most complete that has been offered, and the methods used are applicable to any of the water-sheds of the Sierra Nevada Mountains or, in fact, of other ranges where like conditions exist.

The problem presented is not purely local, as might at first appear. The stream flow from the Sierra Nevada Mountains is one of the greatest factors contributing to the development and prosperity of California, not only because it waters large areas of productive lands in the great central valley, thus making possible large and prosperous agricultural communities, but also because this same stream flow is being utilized by hydro-electric developments which reach to industrial and urban centers far beyond the great valley. During the last twenty to thirty years the U. S. Geological Survey has established stream-gauging stations on the more important streams flowing from the Sierras. In almost every instance, a single gauging station has been established at a point near where the stream emerges from the foot-hills and above most, if not all, diversions for irrigation. Several reasons helped to limit the measurements to this one station on each stream, namely, funds available for the work were limited; the upper basins of these streams were generally in high rugged mountain areas, difficult and expensive of access to hydrographers and often where gauge-height observers were not available (little use was then made of automatic gauge-height recorders); and, finally, the value of these streams for irrigation development was more apparent than the possibility of great hydro-electric developments in the upper basins. Within the past decade, the importance of the possible power developments has become very generally appreciated, and additional stream-gauging stations have been established both by the U. S. Geological Survey, in many instances with State co-operation, and by private interests.

Hence, engineers now charged with the development of hydro-electric power projects are being confronted, almost invariably, with a very short record from the high mountain area in which they are particularly interested, and a much longer record of the flow at a point much farther down stream and for which much of the flow is not available to the project in which they are interested. Obviously, a method of deriving a relation to the flow at the station with the longer record is of the utmost importance. Stream flow for several years in the late Nineties was particularly low, and, except for the last three years, estimates of low flow are usually based on the records for these years.

Until very recently, executive officers and, in many instances, engineers themselves, have not appreciated the necessity of having complete and exact information concerning stream flow before designing large and costly projects. This has resulted in some costly hydrographic blunders where otherwise projects have been carefully designed. In some instances, the potential value of the stream has

* San Francisco, Cal.

not been utilized as completely as economic considerations justify, and, in others, equipment of a greater capacity than was economically justifiable, has been installed, with consequent financial loss.

The method outlined by the author is, as he has noted, as exact as the accuracy of the stream measurements utilized will warrant. Although this is true for Kern River, it is even more evident on other Sierra streams, where one set of measurements is not of the highest degree of accuracy. A recent attempt to apply this method to a study in the San Joaquin River basin was abandoned because of inaccuracies of the shorter record of the upper station. An interesting study of the flow of Merced River by this method is possible because a record of considerable accuracy for the upper station is available from the records of Yosemite Valley stations. For many drainage basins of the Sierra Nevada Mountains, data concerning run-off from the higher areas are all too meager. It is hoped that this condition will not prevail much longer.

H. W. DENNIS,* M. AM. SOC. C. E. (by letter).—The writer is indebted to Mr. Dunn for his discussion. Mr. Dunn suggests a temptation to choose by inspection rather than by analysis the quantity to be used in fixing the size of developments, and it is believed that the method outlined in the paper will provide a rational method of analysis tending to remove such temptation. There will always be erratic variations and apparent discrepancies in stream-flow records, but Mr. Hall's paper (to which Mr. Dunn makes reference) shows by probability curves a method by which such unusual occurrences may be given proper weight in the study of a problem.

The suggestion by Mr. Dunn of the great possibilities of development on the Columbia River emphasizes the desirability of being able to make use of the exceptionally long period of flow records which have been kept at The Dalles, although the points of use may be remote therefrom.

Mr. Dunn's comment on snow surveys is timely, nevertheless, in the writer's opinion, such estimates are valuable only in helping to predict run-off for the season in which they are taken. In general, the time interval between the snowfall and the run-off therefrom is not adequate to permit any change in the amount of plant equipment, but predictions from snow surveys are certainly of great value in arranging the operation of the existing combined hydraulic and steam facilities. This is more especially true in years of apparent deficiency of snow than in years of apparent abundance. The writer believes that a paper on the subject of snow surveys and predictions therefrom would be a desirable addition to engineering data.

The writer is also indebted to Mr. Huber for his discussion, as he gives the historical data leading to present conditions of available stream-flow information.

Although the writer limited his paper to a study of the relation of flow between two points on the same stream, he has been advised that other engineers have since used his method to make a comparison between the flow at two points on different streams. The writer believes this is a reasonable and practicable application of his method, but it is probable that, for satisfactory results, the two different streams under study should have quite similar characteristics as to watershed and exposure.

* Los Angeles, Cal.

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Paper No. 1477.

BRIDGE VERSUS TUNNEL FOR THE PROPOSED HUDSON RIVER CROSSING AT NEW YORK CITY

By J. A. L. Waddell,* M. Am. Soc. C. E.

With Discussion by Messrs. Kennerley Bryan, Charles Evan Fowler, and Shortridge Hardesty.

While making some extensive calculations concerning the costs and economics of long-span suspension bridges for his treatise on "Economics of Bridge-work," the writer has had occasion to figure weights of metal for a number of such spans; and by means of the resulting data he was able to undertake an investigation of the comparative costs and efficiencies of bridges and tunnels for the long-talked-of crossing of the North River at New York City. Thinking that the present is an auspicious time for a thorough discussion of the subject, he has collected and condensed the results of his labors and incorporated them herein for the Society.

For some years the writer has been of the opinion that the best and most economic solution of the problem under consideration is to carry all street cars and subway cars beneath the water and the strictly highway traffic above it. As far as the question of desirability is concerned, this arrangement would be the best practicable for the following reasons:

1.—In respect to cost of operation, the tunnel would require a dip of 90 ft. below high water, and the bridge a rise of 180 ft. above it; consequently, it is evident that, as far as the matter of expenditure of energy is concerned, the tunnel would be decidedly preferable. The difference in cost of power would be very apparent to the management of the electric railways, and, possibly, also to the operators of heavy trucks, but it would not be noticed at all by the owners of automobiles used mainly for pleasure traffic. When an automobilist must climb a long, heavy grade, he seldom considers the extra cost of gasoline; but the officers of an electric railway line generally figure with the greatest care on the item of power expense, and aim to reduce it to a minimum.

2.—In regard to the difference in the expenditures of time in climbing up and down the approaches of the two crossings under comparison, the gravity of

* New York City.

this matter would be duly appreciated by the railroad company and more or less by the operators of trucks, but it would not be recognized by automobile owners and users.

3.—As to the agreeableness of the two kinds of crossing, while people do not particularly fancy going underground before they are ultimately compelled to, they soon become accustomed to passing beneath the water in electric cars, as is evidenced by the many New York business men and women who reside on Long Island or in New Jersey. Perhaps, in time, the drivers of trucks would become so used to traversing tunnels as not to object to the gloom that is inherent in such passage; but the general public, almost to a man (and certainly to a woman), would always greatly prefer driving over a structure that provides good air and light, and usually a fine view of the harbor and the surrounding country, in comparison with traversing a long, cramped, and dingy tube.

4.—In relation to the question of sanitation, there is practically no greater danger to health in passing through a tunnel in which the power used is always electrical than there is in traversing a bridge; but the safe ventilation of a tube carrying automobile traffic is as yet an unsolved problem. Figures show that such ventilation, if feasible, would be exceedingly expensive, and the velocity of the passing air would be excessive.

Again, as carbon monoxide, like arsenic, is a cumulative poison, there exists a possibility that the regular daily passage through a tube where the gas remains constantly, even in minute quantities, would eventually undermine one's health. Besides, there is always the chance of a blockade of traffic with the tunnel full of automobiles, and these may be counted on to discharge more or less products of combustion even when standing still. Such a blockade might result in a holocaust.

The writer sees no serious objection, however, to building the contemplated highway tunnels under the North River; because if, after completion, they prove to be unsafe, or otherwise unsatisfactory for automobile traffic, they can either be used by electric railway cars or else a moving platform can be put in to carry vehicles through without use of their own power. The experiment of automobile transportation through long tunnels might be worth making, for the results in any event would prove of great interest and value to the Engineering Profession, as well as to the general public.

Basis of Cost Estimates.

In making the comparative estimates of cost of bridges and tunnels for this crossing, the writer adopted 4% grades on the approaches of both structures, and clear roadways of 22 ft., with sidewalks 11 ft. wide. He utilized as a basis for his comparison the cost estimates for tunnels given in the report of Clifford M. Holland, M. Am. Soc. C. E., Chief Engineer of the New York State Bridge and Tunnel Commission and the New Jersey Interstate Bridge and Tunnel Commission; but it was necessary to modify some of the items for the increased diameter of tube and the steeper approach grades, also to omit the costs of equipment and administration, as these are not included in the bridge estimates.

For the modification of cost due to a changed diameter of tube, it was assumed that the cost per linear foot increases and decreases as the square of the exterior

diameter. This assumption is almost exactly correct, because the thickness of the shell varies directly with the diameter, as does also the length of the periphery, consequently, the volume of the shell is approximately proportional to the square of the diameter. Again, the volume of excavation for any shield-driven, circular tunnel varies as the square of the diameter; and as almost the entire cost per linear foot is that of the shell plus that of the excavation, it is evident that the assumption mentioned is justified.

Increasing the exterior diameter of the tube to 31 ft. makes the interior diameter 27 ft. This is just sufficient to provide a clear roadway of 22 ft. in the highway tunnel and allows just enough space for two lines of the widest subway cars in the electric-railway tunnel.

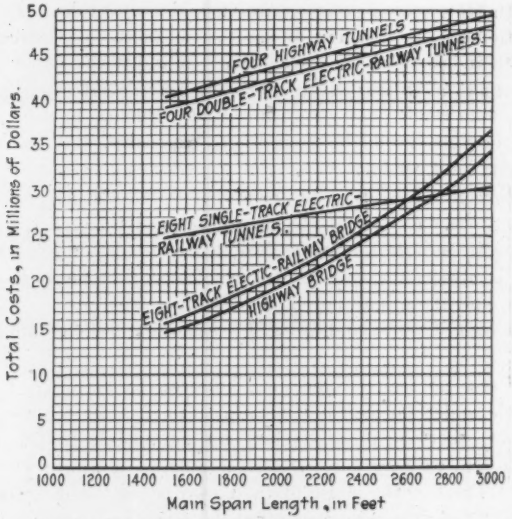

FIG. 1.—DIAGRAM OF TOTAL COSTS OF HIGHWAY AND ELECTRIC RAILWAY BRIDGES AND TUNNELS, WITH THEIR APPROACHES.
For Crossings Similar to That of the North River at New York City.

In the latter type it is feasible to contrast the cost of a double-track tube with that of two single-track tubes; but in a highway tunnel it is not, because a break-down of a single vehicle would block all traffic until it is hauled out by sending a double-ender wrecking-car into the tube in a reverse direction to that of the traffic. With such an occurrence in a double-track tunnel, the automobiles could pass by the wrecked vehicle.

If there was an accidental stoppage of an electric railway car in a single-track tube, it would cause no more obstruction to rail traffic than it would if the accident had taken place in a double-track tube that operates in both directions.

The writer computed all the quantities of materials in substructure, superstructure, and approaches of six structures, in order to plot the two cost curves for bridges shown in Fig. 1. On that diagram are given the total costs for

bridges with their approaches and for tunnels with their approaches, based on the unit prices which governed at the time the tunnel estimates were prepared. The principal unit prices are as follows:

Wire cables in place	$ 0.23 per lb.
Nickel steel in place	0.11 per lb.
Plain concrete in shafts and anchorages	16.00 per cu. yd.
Mass of pneumatic bases	35.00 per cu. yd.

In proportioning the substructures the writer made the dimensions as small as considerations of true efficiency would permit, and did not attempt any beautification of structure by an unnecessary enlargement of piers. If these are properly built to meet all possible conditions of loading and to provide against future deterioration caused by the elements, that ought to suffice and should be deemed good engineering practice.

In order to make the investigation general instead of particular, so as to apply to all similar river crossings by very high-level suspension bridges, the span lengths were assumed to be 1 500, 2 300, and 3 000 ft., to permit the plotting of curves of total costs for both bridges and tunnels, with their approaches, for various widths of river. In this special case the span length would have to be about 2 900 ft.; and in all cases the length of the horizontal portion of the tunnel has been assumed to be exactly equal to that of the main span of the competing bridge.

In the highway-structure comparison, there were adopted for the bridge three clear roadways of 22 ft. each, and two sidewalks of 11 ft. each, corresponding to four double-track tubes each with a clear roadway of 22 ft. The driveways were assumed to consist of creosoted wooden block pavement supported by reinforced concrete slab, and the sidewalks to be of reinforced granitoid.

In the electric-railway structure comparison, the floors were assumed to be of the open type, having wooden ties and guard-rails and the usual steel rails; and eight lines of track were adopted so as to compare with four lines of double-track tubes and with eight lines of single-track tubes.

In every possible manner the comparison was made fair and equitable, except that the costs of right of way and property damages for the approaches, for obvious reasons, had to be ignored. This militated against the tunnel; therefore, in any actual case of comparison, an allowance would have to be made for the difference in cost of right of way and property damages for the approaches to the two structures. In the case of the bridge, by purchasing a large city block close to the water and building a spiral approach thereon, the cost of the said right of way and property damages would be reduced to a minimum; and even that cost might be offset by constructing a high office building above the spiral. Such a building, owing to its location, ought to possess a high rental value.

Fig. 1 shows the total costs of main span, piers, and approaches for both highway and electric railway bridges, and those of the corresponding tunnels.

Relative Costs.

As previously indicated, the shortest span length that can be used for the proposed North River Bridge is about 2 900 ft., for which length Fig. 1 gives approximately the following costs:

Highway bridge	$32 500 000
Four highway tunnels	$49 000 000
Electric-railway bridge	$34 500 000
Four double-track, electric-railway tunnels	$48 000 000
Eight single-track, electric-railway tunnels	$30 000 000

This indicates that there is a saving of cost in favor of the highway bridge amounting to $16 500 000, and one of $4 500 000 in favor of single-track, electric-railway tunnels. The former saving is far greater than the difference in costs of right of way and property damages for bridge and tunnel is ever likely to be; hence, the conclusion is reached that, for the proposed North River crossing, it is not only better from every point of view, but also more economical to carry automobile traffic by a bridge and to carry electric trains by single-track tunnels.

The question arises as to how cheaply there could be built at present prices a combination of bridge and tunnels to care for both kinds of traffic. For a number of years two single-track tubes would take care of the electric-railway trains, hence, the cost would be as follows:

Highway bridge	$32 500 000
Two single-track tunnels	7 500 000
Total	$40 000 000

If it were decided that a bridge having a total width of roadways of 44 ft. and two 11-ft. sidewalks inside the trusses, would suffice for possible future conditions of traffic, the sum required for the combination would reduce to about $33 500 000.

It must not be forgotten that these totals do not include any allowance for right of way, property damages, equipment, or administration.

In conclusion, the writer offers the suggestion that, in view of the facts presented, it might be advisable to give the economics of the proposed crossing of the North River some further study before finally committing the community to the policy that is now contemplated.

DISCUSSION

KENNERLEY BRYAN,* ESQ. (by letter).—The author's suggestion for a division of the traffic between Manhattan and New Jersey, by building tunnels for railway and electric transport, and a bridge for vehicles, certainly deserves serious consideration.

The superior advantage of the tunnel system for railway and electric traffic is unquestionable, especially when considered in connection with a Belt Railway System which must come, and for suburban traffic.

The disadvantages, however, of the tunnel system for vehicular traffic are numerous—length of approaches underground, unless vehicle lifts are installed (which, in themselves, are objectionable), delays due to break-downs of individual vehicles—but, of far greater moment, the possibilities of danger, both to health and from fire, due to the possible accumulation of noxious gases from motor-driven vehicles.

The presence of these noxious gases is certain; their diffusion or extraction by ventilation may be possible, of course, but only at great and continuous expense. Even on a broad, open-air thoroughfare like Fifth Avenue, the objectionable gases ejected by motor vehicles are most noticeable, and, at times, rather overpowering. Unquestionably, they would be injurious to persons forced to inhale them for any length of time, even for so comparatively short a period as that required to traverse a tunnel between Manhattan and New Jersey. This condition being present on an unenclosed avenue, it would certainly be more objectionable in a tunnel, in spite of a reasonably economical ventilating system.

Has this question of the accumulation of noxious gases from motor vehicles been studied from the experience gained in the Glasgow Harbor Tunnel at Glasgow, the Mersey Tunnel at Liverpool, or the Rotherhithe Tunnel under the Thames River at London? Regarding the atmosphere in this latter tunnel, E. J. Mehren, Assoc. Am. Soc. C. E., makes some interesting remarks in his letter published in the *Engineering-News Record* of July 8th, 1920. The removal of vehicular traffic from the tunnel system would obviate the danger, and certainly reduce the initial cost and the cost of operation of the ventilating system.

Dr. Waddell's suggestion for spiral approaches to a bridge is admirable, and entirely feasible, and his remarks about the indifference of motorists, whether on trucks or pleasure cars, as to the time or effort to climb to the bridge level, quite cogent. It is questionable, however, whether Dr. Waddell's suggestion for an office building on top of the spiral approach is quite so good. Rather would the writer suggest warehouses built within the area of the spirals.

It would seem, also, that the views and suggestions made by W. J. Wilgus, M. Am. Soc. C. E., in his interesting paper on "The Railroad Question in Relation to the Metropolitan District",† regarding the possibilities of underground distribution of freight in Manhattan, by means of electric traction direct from the Railway Distributing Yards on the Belt Railway, should have consideration in connection with this problem. In view of the congested area of Manhattan, it strikes one as being desirable to keep the vehicular traffic on or above the surface,

* New York City.

† Read before the New York Section, Am. Soc. C. E., on May 12th, 1920.

leaving the underground and under-water areas free for possible future development for the direct distribution of freight, as suggested by Mr. Wilgus.

Public authority is possibly prone to consider present needs and present expenditure at the expense of foreseen future needs. Is the present problem one which must be solved by the expenditure of a minimum amount of money by the wealthiest communities in the United States, or one which should be solved in a manner which will insure entire safety to the users of the utilities under discussion, at a not unreasonable cost, and which will leave the underground and under-water areas of the country's most congested industrial district free for future development in the most direct and utilitarian manner?

This would seem to be an admirable opportunity for the Society to awaken public opinion to the desirability of a reconsideration of this great problem on the lines of Dr. Waddell's suggestions.

CHARLES EVAN FOWLER,* M. AM. SOC. C. E. (by letter).—The problem of bridge *versus* tunnel for any given crossing involves the study of so many different items that the writer does not believe any direct theoretical comparison will have more than an academic bearing on the solution. Take, for example, a crossing of San Francisco Bay with a distance of approximately six miles to be dealt with, and it is found that a tunnel for any possible route would have to cross a geological fault line and therefore be in danger of destruction from seismic disturbances. The vibrations of a bridge can be provided for in its design. Thus, whatever the results of the economic study, one is inevitably forced to a bridge; while in many locations the cost or availability of right of way, or class of traffic to be handled, would become the governing factors.

The height of the bridge would have a great bearing on the decision, it is true, but the views of the Governmental authorities have undergone considerable change concerning this important question, and they are now disposed to give land and water traffic relatively equal standing. Moreover, recent investigations by the writer have disclosed the fact that masts for wireless apparatus from 150 to 190 ft. high are not necessary, and that wireless equipment is available for any ship having masts upward of 50 ft. in height, with a maximum required limit of 100 ft., for wireless sets of from 2 to 5 kw. Several large shipbuilding concerns also agree with this conclusion, and there would seem to be no need for a bridge clearance greater than 110 ft., except, perhaps, at New York City, where the 135-ft. clearance has become established by the construction of five great bridges over the East River with this head-room. No matter what this height may be, it is really a question of the relative total cost of a probably expensive tunnel approach of comparatively short length, as against the cost of a longer bridge approach at possibly a less total cost, yet giving an equal or less gradient.

There are many engineers to whom it will have to be demonstrated that a tunnel can be ventilated for automobile traffic, even at enormous cost; and it becomes increasingly certain that the cost of obtaining a content of carbon monoxide of even 6 parts in 10 000 will be great. Again, one shudders to contemplate what might happen to those passing through an automobile tunnel in case of a breakdown in the ventilating machinery, even for the short time that might be required for putting into operation a duplicate set. Certain it is that, under the best conditions, it

* New York City.

will be very unpleasant to pass through such a tunnel. The possibility of a holocaust in case of an explosion in an automobile must also receive serious consideration. Is a practical experiment, at so great a cost as nearly $30 000 000, the best that engineers can do in this matter?

There are many other unsolved problems for the automobile tunnel, a serious one being the cleansing of the ventilating ducts of dust, dirt, and oily accretions. These ducts should be not less than 6 ft. 3 in. in height in order to allow workmen to stand upright during the cleansing process, and mechanical cleansers must be designed to do the work expeditiously, with tracks and cars or conveyors to remove the accretions. This would seem to call for an increase in tube diameter of from 3 to 4 ft., or for a total diameter of not less than 32 ft. Then, suppose that eventually the tunnel must be turned over to electric traction, the clear height should be not less than 14 ft. and, preferably, 16 ft., thus increasing the total diameter to about 35 ft. Assuming that the width of roadway of 22 ft. is sufficient —which must be seriously questioned—then it is possible to design a tunnel of a vertical elliptical section to give the necessary increased height, and account for the extra side pressure by struts at the floor and ceiling lines.

Before leaving the problem of the tunnel, it is well to consider the open question of whether to draw the foul and poisonous air from the bottom or the top. The only logical solution would seem to be to draw it out below the floor and thus get it away quickly from the point of production, at the exhaust of the automobiles, and not have it pass the faces of passengers or horses. This, it is true, might add to the cost of cleansing the ducts.

Taking it all in all, there are many problems that affect the cost of a tunnel which do not have to be taken into account with a bridge, even if a spiral roadway should be used, which, in the opinion of many engineers, would be a very bad feature and one that would certainly cause frequent traffic congestion.

The paramount reasons for a bridge across the Hudson River are those of comfort, safety, and pleasure for the passenger, and the more vital one of a need for handling the railway freight traffic to and across Manhattan Island. No tunnel or tunnels can provide the necessary freight terminals, as underground yards would be prohibitive in cost and in cost of enlargement. In the case of a bridge the elevated approaches could be provided with frequent sidings, at a comparatively small cost, and be extended to reach several or many surface yards.

The study of any particular crossing will certainly entail the consideration of many problems in addition to the ones thus far discussed, and complete designs and estimates must be made for each individual case, unless, as has been indicated, particular conditions point to an inevitable conclusion.

SHORTRIDGE HARDESTY,* ASSOC. M. AM. SOC. C. E. (by letter).†—In their discussions, Mr. Bryan and Mr. Fowler both emphasize Dr. Waddell's chief objections to vehicular tunnels, namely, the uncertainty concerning their proper ventilation, and the unpleasantness of traversing them. Certainly, such tunnels should rarely be considered where a bridge is practicable.

The paper deals mainly with the relative costs and desirability of the bridge and tunnel structures. The cost of real estate is an important factor which

* New York City.

† In the absence of the author, J. A. L. Waddell, M. Am. Soc. C. E., in the Far East, this closing discussion has been prepared for him by Mr. Hardesty.

usually favors the tunnel; and, as suggested by Mr. Fowler, other general considerations, such as the handling of freight and passenger traffic at both termini, will frequently be governing conditions for any particular case.

While completing the preparation of Dr. Waddell's discussion on "Bridge *versus* Tunnel", for the meeting of the New York Section of the American Society of Civil Engineers, on May 11th, 1921, the writer revised Fig. 1 of the paper by plotting the curve for the highway bridge for 88 ft. of roadway instead of 66 ft. of roadway and 22 ft. of sidewalk, omitting the curve for four double-track electric railway tunnels, and adding curves for an eight-track standard railway

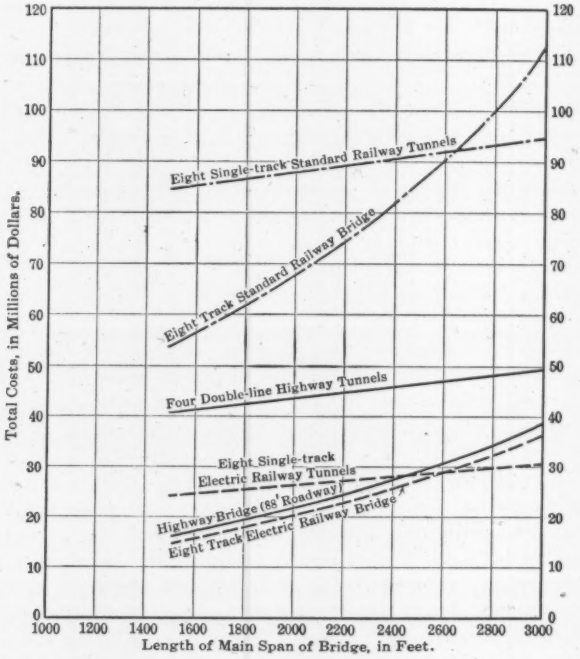

FIG. 2.

bridge and eight single-track standard railway tunnels. The revised diagram is shown herewith as Fig. 2. This diagram contrasts the costs of the following structures: four double-line highway tunnels with outside diameters of 31 ft.; eight single-track electric railway tunnels with outside diameters of 17 ft.; eight single-track standard railway tunnels with outside diameters of 23 ft. 9 in.; an eight-lane highway bridge; an eight-track electric railway bridge; and an eight-track standard railway bridge.

The following general conclusions can be drawn from Fig. 2: The highway and electric railway bridges cost about the same for all span lengths, while the railway bridge costs much more than either of the other bridges. This is

due partly to the greater live load to be carried, and partly to the greater length of the approaches.

The electric railway tunnels are much cheaper than either of the others, because of their smaller diameter. The four highway tunnels cost nearly as much per linear foot as the eight standard railway tunnels; but the total cost of the latter is much greater on account of the longer approaches.

Increasing the span length increases the cost of a bridge much more rapidly than that of a tunnel. Short span lengths, therefore, tend to favor a bridge, and long span lengths a tunnel.

The bridge is more likely to be economic for highway traffic than for either of the other traffics. The tunnel is about equally economic for railway traffic and electric railway traffic.

When property damages are taken into consideration, railway or electric railway tunnels for spans much exceeding 2 000 ft. will usually be cheaper than railway or electric railway bridges.

The highway bridge would appear to be cheaper than the highway tunnel even for a 3 000-ft. span, unless the property damages are quite heavy.

It must not be forgotten that the foregoing costs cover only one phase of this problem; and that they may be modified appreciably by various practical considerations at any particular location, such as the elevation of the ground on each side of the river. The tunnels possess an economic advantage in that they can be built when and where they are needed.

If provision must be made for a small number of lines of traffic only, this condition favors the tunnel solution decidedly, especially when a very long span is required for the bridge. On the other hand, a great number of lines of traffic will tend to make a bridge more economic.

The cost estimated for the great double-deck bridge proposed by Gustav Lindenthal, M. Am. Soc. C. E., for crossing the Hudson River at New York City, may be less than a number of tunnels sufficient to provide the same capacity, on account of the great number of lines of traffic provided for. It is the writer's opinion however, that, when property damages for the long railway approaches are duly considered, it will be found to be cheaper to carry all the tracks in tunnels, leaving the bridge to carry highway traffic only. Plans for the handling of traffic at both the New York and New Jersey ends must be workeed out completely for both schemes before any final decision is possible.

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Paper No. 1478

VERTICAL LIFT BRIDGES

By Ernest E. Howard,* M. Am. Soc. C. E.

With Discussion by Messrs. D. B. Steinman, Shortridge Hardesty, Thomas E. Brown, G. Lindenthal, Arthur G. Hayden, Harry G. Hunter, E. G. Walker, Horatio P. Van Cleve, F. W. Green, Victor H. Cochrane, Henry C. Tammen, and Ernest E. Howard.

Synopsis.

The purpose of this paper is to outline the development of the vertical lift bridge, to discuss its principal elements, and to describe noteworthy features of some structures in which it is used. Part I deals with the general elements of lift-bridge design; Part II describes a bridge with a lifting span, the Columbia River Interstate Bridge; Part III describes a bridge with a lifting deck, the North Kansas City Bridge; and Part IV describes a bridge with a lifting span which has a lifting deck, the Harriman Bridge over the Willamette River at Portland, Ore.

Twenty-five years ago the swing span was almost the only type of movable span in use, but its limitations and deficiencies brought engineers to devise spans which would move vertically instead of horizontally. These include many ingenious bascule bridges which rotate in vertical planes, and the lift bridge which moves vertically. Although a rare type twelve years ago, more than forty lift spans have since been built, many in large and important structures. Since only fragmentary information about these structures has hitherto been available, the writer undertakes to describe in some detail the essential features of the foregoing bridges, as containing elements which typify the development of the lift-bridge idea, and as showing adaptability to widely varying conditions and limitations.

The paper indicates the advantages of the lift bridge—economy in construction, rapid operation at low cost, determinate stresses, and no new stress conditions introduced during operation. It may within reason be of any length, any width, of any material, and with any type of floor and pavement. Future alterations in the lifting span, or its floor, and future grade changes are readily allowed for; and the effect of wind on operating machinery is very slight. It is shown that few engineering elements are more reliable, efficient, and widely used than wire ropes, and most important lift spans are suspended by such ropes, although built-up chains, wrought chains, systems of levers, etc., have been considered and even tried. For counterweights for lift bridges cast iron has been used, but concrete

Presented at the meeting of May 4th, 1921.

* Kansas City, Mo.

built around a steel framework is commonly used as it costs less than one-third as much as cast iron.

The paper illustrates the operating machinery—ordinary hoisting equipment, simple and easily adjusted—and indicates that simple spur gearing ordinarily suffices, the longest shaft extending transversely from truss to truss. Electric or hand brakes, limit switches for automatic emergency stops, gasoline engines or electric motors, suitable position indicators, etc., are used.

Direct comparison of the operation of various types of movable spans in which the lift span is found to operate more quickly and costs less per operation than adjacent swing and bascule bridges, is made, speed of operation being due in part to the fact that the lift span need only be lifted high enough to pass the approaching boat (often only 15 or 20 ft.), and that there are no wedges or locks to be driven after the span is seated.

The Columbia River Interstate Bridge described in Part II includes a simple lift span 275 ft. long, 50 ft. wide, with a concrete floor and sidewalk, which will lift to a clear height of 150 ft. Two others of the twenty-nine spans of the structure are arranged to be converted into lift spans by the erection of towers, counterweights, machinery, etc., should increased navigation require it. The somewhat unusual methods of pier construction, superstructure erection, pumping embankment approaches, floor details, low construction costs, and the public acquirement of the property without taxation are notable in this bridge.

It is shown that the lift-bridge idea may be applied not only to lifting spans, but also to a bridge floor supported at several points along its length, called a lifting deck, Part III, which is peculiarly adaptable to double-deck structures. In older forms, of which a few small spans have been built, the lifting deck and its counterweights were supported from the overhead trusses. The later structures, as described, relieve the supporting trusses of about half this load by placing the counterweights at the ends of the span. The North Kansas City Bridge over the Missouri River has a lifting deck, 425 ft. long, serving a double-track railway. The special problems presented by the conditions and satisfactorily solved by the lifting deck are described.

The Harriman Bridge over the Willamette River at Portland, Ore., described in Part IV, offers a noteworthy example of the exceedingly feasible variations possible with the wire-rope lift-bridge idea, for it has a lifting deck suspended under a lifting span which can be operated without movement of the span; or both can be raised at will. This design met the special condition of dense highway traffic on an upper level which is obstructed only a few times a day for the passage of masted vessels, although the lower deck is open sometimes for as many as 100 boats per day. The river having a maximum depth of 90 ft., the piers extend to depths exceeding 130 ft., involving unusual construction methods which are described.

Quantities and costs of these three special bridges are included, and Table 1 gives general data of the more important lift bridges that have been built.

PART I.—GENERAL ELEMENTS OF DESIGN.

The lift bridge is of ancient origin. Several small lift spans were built in Europe prior to 1850. Concurrently in the United States similar designs, or inven-

tions, were made, as lift-bridge patents were granted at early dates to Messrs. McNeal, who secured a broad basic patent, Post, Whittemore, and the late Squire Whipple, Hon. M. Am. Soc. C. E. Lift bridges were occasionally proposed, but none more important than canal crossings was built until, in 1893, J. A. L. Waddell, M. Am. Soc. C. E., designed the bridge at Halsted Street, Chicago, Ill., over the Chicago River, which is still in service.* After that time a few small bridges were built over the Erie Canal, but 15 years elapsed before the idea was applied to any large structures, when the development herein discussed began.

The principal element of the lift bridge, in its simplest form, is a simple span equipped with machinery for operation, suspended at each end by wire ropes which pass over sheaves on towers and connect to counterweights about equal to the span weight. Variations in almost every feature have developed: Towers may be in part supported on adjacent spans; four vertical columns per tower, with four sheaves, may be used; both towers and span may be square or skewed in plan.

When the lift span is short, the rear bracing of the towers may be omitted and overhead struts used instead. Where the lift movement is small, each column may be independent, the sheaves placed transverse to the span, and two counterweights hung outside the columns at each end. In one bridge the columns are part of the lift span, and extend down as legs into wells in the piers; the sheaves are on the piers and the counterweights are suspended underneath the floor, thus no tower or machinery is above the deck. Towers may be of steel, wood, or concrete; concrete towers may be treated architecturally and excellent appearance secured.

Adaptability and Economy.

In some alluvial rivers the location of deep-water channels varies through periods of years. Large expenditures occasionally have been made to construct additional movable spans or to maintain a channel under an existing span. This condition may be met by the use of equal spans, as any span can be made to lift and the towers supported on adjacent spans; towers and machinery may be removed from one location and re-erected in another. Similarly, a bridge may be first constructed with fixed spans, and, later, one span may be made movable; also, towers and machinery may be installed and an old fixed span converted into a lift span, thus economically providing for navigation.

Substructures as well as superstructures must be considered in determining comparative economics of movable spans. Tower piers of lift bridges differ little from ordinary piers. The added loads are vertical and do not change in amount or position during span movement, wind excepted. The elaborate protection pier, common with the swing span, is not necessary, and the lift span shows its greatest comparative economy where foundations are deep or expensive, or are carried on piles. For narrow openings where vessels with lofty masts must be accommodated a bascule is preferable; but where the vertical clearance required does not exceed the channel width, the lift span is found to be nearly always cheaper than the swing bridge or bascule, notably for long spans.

Qualities of Wire Rope.

The qualities of wire rope are essential features of lift-bridge development. A wire rope composed of steel wires twisted into strands about a central rope or strand

* *Transactions*, Am. Soc. C. E., Vol. XXXIII (1895), p. 1.

of wires or oil-saturated hemp, is much more elastic than the elements composing it and more flexible than a bundle of the same wires wrapped together straight. Extreme fiber stresses from bends of large radius are very small. Steel of great tensile strength can be used, and no other flexible tension member can approach so small a cross-section with equal strength. Each wire and the finished rope can be tested and an element of marked homogeneity secured of varying characteristics as desired.

The life of a wire rope operating over a sheave is a function of the number of operations. Biggart's experiments* with sheaves up to 42 rope diameters developed that the first wire will break approximately at the mid-point of rope life in operations. Applied to sheaves of 60 diameters, his formulas indicate a rope life of about 1 000 000 operations, or for a bridge, say, 50 operations daily up and down for 25 years. Such rope life appears to be extreme, but the Halsted Street Bridge has been in service 27 years and has operated probably 200 000 rope bendings. The main suspending ropes originally placed are still in service, and when last examined by the writer no indications of failure were found. Yearly examination of suspending ropes will give timely warning of approaching weakness long before replacement is necessary. With suitable connections such replacement can be made without interference with traffic or bridge movement.

Forged steel sockets of full rope strength are used to connect ropes to structural parts. The load on a group of ropes of unequal length or uneven stretch may be distributed equally to each by equalizing bars or levers. A clamp on the ropes just below the sheave prevents movements of equalizers due to rope divergence, yet leaves every rope free to equalize.

Suspending Sheaves.

Suspending sheaves of 60 to 80 rope diameters afford satisfactory proportions between direct tension and bending stresses in ropes. Grooved sheaves of single steel castings, or of cast rim sections and hub riveted to structural webs are used. Composite sheaves not fully riveted are not satisfactory unless the shop work is better than is ordinarily the case. Sheaves 12 ft. in diameter, 36-in. face, have been supplied as single castings.

The two sheaves on a tower are quite independent, and slight variations in position cause no difficulty provided only that each pair of bearings are in proper relation. The sheaves can be skewed and the suspending ropes led directly from lift-span trusses to counterweight, or the sheaves can be set in any lateral position and the ropes connected to a portal girder of the lift span.

Operating Machinery.

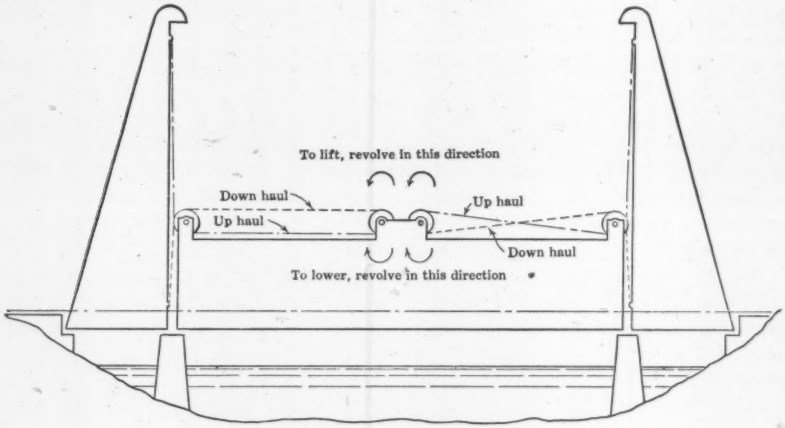
Operation can be effected by causing the tower sheaves to revolve synchronously by connecting shafts or similar mechanisms, or operating drums and machinery may be placed in or under one tower. Rack and pinion operation is possible for small spans, but has not been used for any large ones.

The most used and most approved operating machinery comprises operating drums with motor attachment on the lift span controlling operating ropes, Fig. 1. Near the middle of the span are four spirally grooved drums connected through trains of gears to motor or engine. Each drum contains and controls two operat-

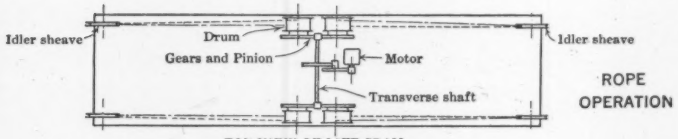
* Biggart on "Wire Ropes," *Minutes of Proceedings*, Inst. C. E., Vol. CI, p. 231.

ing ropes which pass, respectively, under and over deflecting sheaves at the corner of the span, thence one upward and one downward to connections on the tower column. Revolution in one direction winds on the up-haul ropes, pays off the down-haul ropes, and lifts the span; reversal of the drums pulls the span down.

The operating ropes connect to take-up devices and may be adjusted to have the same general tension at each corner. The span is thus moved, held level during operation, and may be held at any point or locked down by stopping the machinery. No other locks are necessary, although occasionally used, principally as part of a track-interlocking system.



ELEVATION OF LIFT SPAN SHOWING OPERATING ROPES
(Suspending ropes and counterweights omitted)



TOP VIEW OF LIFT SPAN

FIG. 1.

CONSTRUCTION OF COUNTERWEIGHTS.

Methods of constructing counterweights vary with navigation conditions. When there is a closed season the lift span is placed on the piers, the suspending ropes connected and laid over the sheaves, the counterweight frames hung in place, wood forms bolted to them, and the concrete deposited. When the channel must be kept clear, the counterweights can be built in raised position on special beams attached to the towers or other falsework, the lift span floated in, attached to the ropes, and lowered to lift the counterweights so their supports may be removed; or the lift span may be erected in elevated position.

Counterweights of pre-cast blocks can be erected with ordinary equipment without falsework, provided only that the channel may be blocked for the few hours needed to raise and place the counterweight parts. They can be taken down

TABLE 1.—LIFT-BRIDGE DATA.

Bridge over:	Kind of traffic.	Length of lift span, in feet.	Maximum clear height above high water, in feet.	Normal height of movement, in feet.	Total moving load—span counterweights, etc.—in tons.
Arkansas River near Ft. Smith, Ark., for Ft. Smith-Van Buren Bridge Company.	Railway, street-car, and highway.	192	55	50	1 599
Arkansas River near Watson, Ark., for Missouri Pacific System (M. H. & L. Railway).	Single-track railway.	208	60	55	786
Bray's Bayou at Harrisburg, Tex., for Harris County.	Highway.	48	26	10	346
Bray's Bayou at Harrisburg, Tex., for Southern Pacific System.	Single-track railway.	48	20	17	54
Black River near Jonesville, La., for Louisiana and Arkansas Railway Company.	Single-track railway.	165	55	53	579
Big Choctaw Bayou, Southern Louisiana, for Southern Pacific System.	Single-track railway.	50	47	40	71
Big Warrior River at Cordova, Ala., for Frisco System.	Single-track railway.	184	26	22	789
Caddo Lake at Mooringsport, La., for Caddo Parish.	Highway.	92	55	49	223
Calumet River at South Chicago, Ill., for Pennsylvania System. (Two bridges.)	Double-track railway.	210	120	98	2 052
Calumet River at South Chicago, Ill., for New York Central (L. S. & M. S.) (Two bridges.)	Double-track railway.	210	120	98	1 784
Chicago River at South Halsted St., Chicago, for City of Chicago.	Highway and street-car.	130	155	142	660
Chicago River at Chicago for Pennsylvania System.	Double-track railway.	273	120	111	3 437
City Waterway at Tacoma, Wash., for City of Tacoma.	Highway and street-car.	214	135	75	1 677
Columbia River at Trail B. C., for Province of British Columbia.	Highway.	171	55	47	271
Columbia River, Vancouver, Wash., to Portland, Ore., for Multnomah, Oregon and Clarke Counties, Wash.	Highway and street-car.	272	150	136	2 581
Don River No. 1, at Rostoff, Russia, for Vladicaucase Railway Company.	Double-track railway.	210	140	127	1 694
Don River No. 2, at Rostoff, Russia, for Vladicaucase Railway Company.	Single-track railway.	210	140	127	980
Flint River near Newton, Ga., for Baker and Mitchell Counties.	Highway.	87	35	30	90
Illinois River near Pekin, Ill., for Chicago Northwestern System.	Single-track railway.	173	55	39	785
Little River near Jonesville, La., for Louisiana and Arkansas Railway.	Single-track railway.	118	45	41	396
Mississippi River at St. Paul, Minn., for Chicago Great Western Railway.	Single-track railway.	189	58	53	880
Mississippi River at Keithsburg, Ill., for Iowa Central Railway Company.	Single-track railway.	230	55	45	980
Missouri River near Snowden, Mont., for Great Northern System.	Single-track railway.	296	50	40	1 621

TABLE 1.—(*Continued*).

Bridge over:	Kind of traffic.	Length of lift span, in feet.	Maximum clear height above high water, in feet.	Normal height of movement, in feet.	Total moving load—span, counterweights, etc.—in tons.
Missouri River at Kansas City, Mo., for Union Depot and Terminal Railway Company.	Double-track railway, street-car and highway, double-deck.	426	65	45	1 595
Missouri River at Yankton, S. Dak., for Meridian Highway Bridge Company.	Single-track railway and highway.	246	38	31	1 762
Oromocto River near Fredericton, N. B., for St. John and Quebec Railway.	Single-track railway.	58	78	56	152
Puyallup River at Tacoma, Wash., for City of Tacoma.	Highway and street-car.	161	135	112	982
Red River near Index, Tex., for Red River Bridge District.	Highway.	200	55	50	825
Sacramento River at Tehama, Cal., for Tehama County.	Highway.	167	65	59	271
St. Francis River near Memphis, Tenn., for Missouri Pacific System.	Single-track railway.	162	70	68	582
St. Johns River at Jacksonville, Fla., for Duvall County.	Highway and street-car.	188	165	108	1 458
Thompson River near Kamloops, B. C., for Canadian Northern Railway.	Single-track railway.	90	55	53	219
Willamette River at Portland, Ore., for City of Portland.	Highway and street-car.	244	135	110	1 825
Willamette River at Portland, Ore., for Harriman System (O. W. R. & N. Co.)	Double-track railway, street-car and highway, double-deck.	UD 211 LD 211	140 140	89 46–89	3 575 1 010 4 585
Willamette River at Harrisburg, Ore., for Northern Pacific.	Single-track railway.	200	70	60	678
Willamette River at Salem, Ore., for Southern Pacific System.	Single-track railway.	131	55	51	479
Yellowstone River near Fairview, Mont., for Great Northern System.	Single-track railway.	271	50	40	1 423

as readily, so this form is always used where various spans are made liftable, and is regarded with favor for any bridge.

Except when the span and counterweights are at the same elevation, the suspending ropes are unbalanced. Ordinarily, this varying unbalanced load is handled by the surplus capacity of the operating machinery. Exact balance for all conditions may be secured with counterbalancing chains. Suspending sheaves may be used at the corners of a lift span effecting a three-part rigging of suspending ropes, reducing the counterweights by one-half, and doubling their height of movement. This was found advantageous with a very heavy span with low lift.

The counterweights are guided during movement by jaws engaging vertical tracks on the towers. The span is similarly guided, with allowances for temperature variations of span length, and for the deflections of the towers. At one end the span is held both laterally and longitudinally, at the other only laterally. To bring the span to exact lateral position for proper junction of track rails, the lower few feet of the guide-tracks are battered out to take up all lateral clearance

FIG. 2.—COLUMBIA RIVER INTERSTATE BRIDGE: GENERAL VIEW FROM VANCOUVER END.

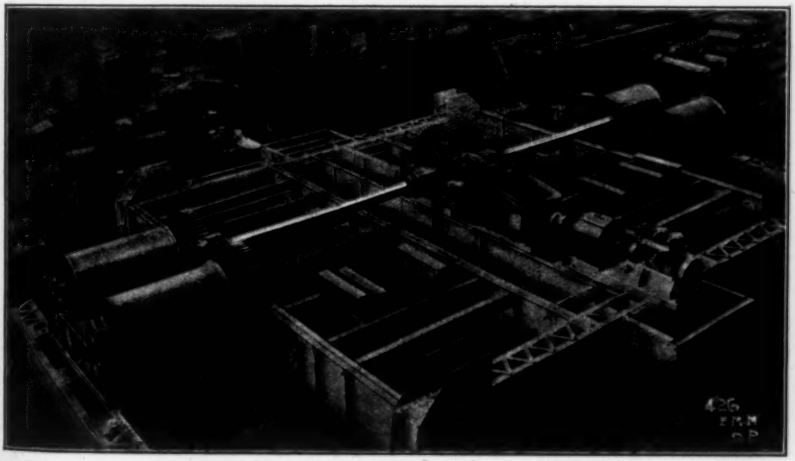


FIG. 3.—OPERATING MACHINERY, COLUMBIA RIVER INTERSTATE BRIDGE.

in the guides at the seated position of the span. The guide-jaws may have rollers. Spring guides bearing on tracks on the faces of tower columns have been used, but are less satisfactory than the jaw guides.

The pier at the end of a lifting span supports, in addition to the loads of a fixed span, one tower, sheaves, and ropes, one counterweight, and additional wind load. For a single-track railroad bridge, Class E-55 loading, and lift span 200 ft. long for an ordinary inland river, this total extra load on one pier, except wind, is about 275 tons, which is additional to an ordinary imposed load for the fixed span of about 1 200 tons.

Near together over the Willamette River in Portland, Ore., serving the same kind of traffic, are one steam operated swing span, one electrically operated swing span, one excellent double-leaf bascule span, one double-deck lift span, and one simple lift span. County records show that the latter, the Hawthorne Avenue Bridge, serves more tonnage of traffic, operates more quickly, and costs less per operation than any of the others. Its openings average 15 daily, the average time that traffic is delayed per opening is 110 sec., the power consumption is less than 2 kw. per operation, and the cost of power per operation is about 5 cents. The lift span is 245 ft. long.

Many engineers for the owners of lift bridges and of the staffs of contractors and manufacturers have been identified with the design and construction of the bridges as listed in Table 1. The development from 1909 to 1914 was principally by the firm of Waddell and Harrington, of which the writer was Associate Engineer, and since 1914 largely by the writer's present firm of Harrington, Howard and Ash.

PART II.—A BRIDGE WITH A LIFT SPAN.

The Columbia River Interstate Bridge.

The Columbia River Interstate Bridge, Fig. 2, with its approaches, extends across the valley of the Columbia River from Vancouver, Wash., to Portland, Ore., a distance of 3¼ miles, and includes about 5 000 lin. ft. of steel bridge structures and 12 000 lin. ft. of embankment, in addition to which there is a secondary approach embankment on the Oregon side about 6 000 ft. long. The bridge and approaches provide a roadway for street traffic and tracks for street cars, and a sidewalk over the steel structures.

Clarke County, Washington, and Multnomah County, Oregon, built the structure with funds from issues of bonds, and operate it as a toll bridge. The construction cost was about $1 700 000; the gross receipts, beginning February, 1917, when it was opened to traffic, were about $300 per day, but increased rapidly and for the past few years have approximated $300 000 annually. Within a few years the counties will have retired the bonds and will own the structure unencumbered, acquired without any taxation. This is the only highway bridge across the Columbia River for a distance of more than 300 miles from the Pacific Ocean.

The Columbia River at this site has a maximum depth of 30 ft. The normal spring rise is 20 ft.; about once in four years it is 25 ft., and once it reached 33 ft. At extreme flood it covers the entire valley. The river bed is of sand, with some gravel, to great depths. Hard conglomerate—cemented gravel—is found near the surface at the north shore, but drops rapidly to a depth of 100 ft. at 700 ft. from

shore. Oregon Slough, a secondary channel separated from the main river by Hayden Island, is about 25 ft. deep, with a bed of sand to great depths. Columbia Slough provides some drainage near the north side of the valley, and is about 20 ft. deep. Hard material is found 80 ft. below water, overlaid with soft clay and sand. The average ground elevation over the valley is 12 to 15 ft. above low water.

Over the Columbia River are three 275-ft. through-riveted truss spans with curved top chords, ten similar spans 265 ft. long, and one small deck girder span, making a total length between end shoes of 3 531 ft. 6 in. The central of the three 275-ft. spans is arranged to lift between towers on the other two, affording for navigation a channel 250 ft. wide with a vertical clearance of 150 ft. at high water and 175 ft. at low water. To date, no other movable span constructed with concrete floor and hard surface pavement provides so wide a channel.

Over Oregon Slough are ten deck girder spans 100 ft. long and one similar span 115 ft. long, a total length of 1 137 ft. 6 in. There is now no navigation in Oregon Slough above the bridge site. The War Department permitted construction with all fixed spans, with the provision that one span should be convertible into a movable span. The 115-ft. span so converted, by constructing towers at each end, providing machinery, counterweights, etc., will afford a channel 100 ft. wide. This arrangement saved a present expenditure of about $30 000, and added practically no cost to the present structure. Future construction will not interrupt traffic, and will not require any parts now built to be discarded. Over the Columbia Slough, there are four deck girder spans 75 ft. long. Future navigation can here be similarly accommodated.

The roadway over the steel spans and on Hayden Island is 38 ft. wide, on Union Avenue Approach 30 ft. wide, and on Derby Street Approach 24 ft. wide. The embankments are generally about 42 ft. wide and 24 ft. high, with 2 to 1 side slopes.

The structure is designed to carry 50-ton street cars, motor trucks of 24 tons maximum, roadway loads of 500 to 800 lb. per lin. ft. per line of traffic, and suitable sidewalk loads.

Total Quantities.

The approximate quantities in the structure are as follows:

Superstructure:	
Truss spans and girder spans	8 073 tons
Towers and counterweights	412 "
Machinery and wire ropes	178 "
Street-car rails and fastenings	440 short tons
Roadway slab and pavement on spans	21 610 sq. yd.
Sidewalk slab	3 250 " "
Substructure and Miscellaneous:	
Volume of piers	28 050 cu. yd.
Piles below bases	133 500 lin. ft.
Reinforcing metal in piers	70 tons
Rip-rap stone and rock fills	23 300 cu. yd.
Embankment	1 475 000 " "

Concrete slope protection on embankment	51 200 sq. yd.
Pavement base (broken stone)	16 875 cu. yd.
Bitulithic pavement on embankments	45 200 sq. yd.

The time of construction was two years.

Substructure.

The facility with which extremely long timber piles can be secured in this district at low price was a factor in the pier design. The river is subject to some scour, but soundings taken over many years disclosed no indications of scour to extreme or unusual depths at this site. The bases of the piers are of concrete enclosed by heavy timber cribs, and their bottoms are 20 to 25 ft. below the river bed. They rest on piles which extend down about 110 ft. below low water, and up into the bases 15 to 20 ft. The piles are thus thoroughly embedded in the concrete and form definite parts of the piers; they are single sticks about 120 ft. long, with 10-in. tips and 22-in. butts. There is an average of 1 pile to 10 sq. ft. of area, and in the ordinary pier under 265-ft. spans there are 90 piles. The total load per pile, dead and live, is about 32 tons. Around each pier base there is heavy stone riprapping which will be replenished if necessary. Even though some unusual and extraordinary scour should extend below the bottom of its base, a pier would still be wholly stable because of the great penetration of the piles and the rigidity of their upper embedment.

The pier shafts are of concrete, each shaft comprising a pair of cylindrical battered columns, joined by a vertical diaphragm and by a horizontal coping at the top, both reinforced. Large floes or sheets of ice in the Columbia River are rare and the concrete piers need no special protection of stone or steel facing.

The piers were built by excavating a hole in the river bed to the full depth required, large enough for the sides to take a natural slope. A dipper dredge, of shovel type with 4-yd. bucket, was used and the excavated material taken by barge and used in embankments. The base crib was partly built on launching ways, floated to position and framed to full height. A removable coffer-dam in one section, used successively on different piers, was then placed on the crib, and the whole sunk to proper depth and material banked outside. The piles were then sunk inside the crib by water-jets.

For sinking the piles a driver was provided with leads 122 ft. high and built so as to overhang the barge supporting it about 22 ft. The bottom of this overhanging part was 15 ft. above water, so that it was possible to drive all the piles in a pier from one side of the crib and at any stage of water. The pile-driver was equipped with a heavy steam hammer and with two 4-in. diameter jet pipes, 110 ft. long, connected by hose to the pumps. Two 12 by 18 by 10-in., compound duplex pumps, each with a capacity of 1 000 gal. per min., were used for jetting. One four-drum and one two-drum engine handled the equipment, and steam was supplied from one 800-h. p. boiler. With jets and hammer working, 10 min. sufficed to sink a pile to full depth, and as many as 25 piles were sunk per day.

After the piles were driven, the material washed into the crib by the jetting was pumped out and the base filled by depositing concrete under water through a tremie up to within a few feet below water. The crib was later pumped out, the piles were cut off, and the remainder of the concrete was placed in the open. The

one-section coffer-dam was lifted off after the lowest section of the shaft had been concreted, and the remainder of the shaft forms and concrete placed. The pier was then back-filled and rip-rapped.

Superstructure.

The trusses of the through spans are 41 ft., center to center, with the roadway between and the sidewalk outside one truss. The floor is a reinforced concrete slab, 5½ in. thick; the sidewalk also is of reinforced concrete. The floor arrangement is new. Between the usual floor-beams are longitudinal stringers about 9 ft., center to center. On top of the stringers, extending across the roadway, are 8-in. I-beams about 33 in. apart. These beams are bent to conform to the crown of the roadway, and support the concrete slab. The six rails for the two-gauge, double-track street-car tracks also are laid on these beams and bolted to them. The rails are 7 in. high, allowing depth for the 2-in. bitulithic pavement surfacing. A space about 5 in. wide on each side of each rail is filled with rich concrete for the full rail depth and, on the gauge side, forward for a flangeway. These concrete rail-headings were compacted into place and the flangeways formed by rolling the plastic material with specially made cast-iron wheels.

Two fixed ends and two expansion ends of the spans are adjacent, to reduce the number of expansion joints. The joints for the rails between spans are of manganese steel castings placed against the rail webs, for which one side of the rail-heads is planed off.

For the movable span, both a lift and a swing span were considered. Tentative approval was received from the War Department for a swing span affording two channels of 200 ft., and for a lift span for one channel of 250 ft., with vertical clearance 150 ft. above normal high water. Complete plans were made for both structures and bids received for both. The lowest bid for the bridge including the lift span was $70 000 less than the lowest bid for the bridge with a swing span. The lift also causes the least obstruction to river traffic. The main channel is close to the Vancouver shore and the necessary draw protection, about 500 ft. long for a swing span, would have been a serious detriment to the dock frontage adjacent to the bridge. No fenders have been found necessary with the lift span.

Arrangement of Machinery.

The arrangement of lift, towers, and machinery is characteristic of the latest practice. One transverse shaft, one pair of gears in the frame inside the house, the back gears of the motors, the drums, gears, and frames, and the corner deflecting sheaves make up the entire operating machinery (Fig. 3). There are two motors, and for emergency operation a 12-h. p. gasoline engine is connected through a speed reducer. The suspended load is 1 200 tons, and the total moving load 2 500 tons.

The counterweights may descend to within 2 ft. of the floor level, and to allow this movement the trolley wires under the counterweights are fastened to swinging frames which are pushed down by the counterweights and returned to position by small counterbalances. Manually operated roadway gates are located near the ends of the lift span, and operate derailing switches in the street-car tracks.

Erection.

Erection of the main river bridge was accomplished principally by floating from an erection and material yard in Vancouver. Each span in turn was erected parallel to the river on low falsework well back on the bank at the pier elevation. Timber trestles extended from it out into deep water, one at each end of the span, and constituted launching ways. After each span was completed and all rivets were driven, it was rolled out on the ways over deep water.

Four barges surmounted by timber falsework were used to carry the spans to the piers. Variation of height for lifting and landing the spans was obtained by pumping water into and out of these barges. The lift span was floated to its position the same as the other spans. The span second beyond it was not placed until the towers and machinery had been installed and the lift span made operative, thus affording a channel for navigation. The towers were erected with a derrick on a wooden tower built on top of the lift span. The suspending ropes were attached to the lift span and laid over the sheaves and the counterweight frames suspended. Wood forms were bolted to these frames and the concrete elevated and placed in the high position of the counterweights. In due time the forms were removed, the operating ropes connected to the machinery, and the whole adjusted for operation.

Embankment Construction.

The approach embankments are of sand, principally dredged from Oregon Slough by a suction dredge with cutting head, and pumped through 24-in. pipe. The pump on the dredge was operated by two 500-h. p. motors, and was of capacity to give a discharge through the 24-in. pipe at velocities of 12 to 15 ft. per sec. Operation was 24 hours per day, and the actual net time operated was 14 hours per day, which placed in the embankment in addition to wastage an average of 5 000 cu. yd. net per day. As much as 1 000 cu. yd. per hour was placed for a few hours at a time. The discharge pipe was extended 5 500 ft., working from the dredge alone. A booster pump was installed, operated by one 1 000-h. p. motor, and the material handled through a total length of 9 000 ft. of pipe.

The embankment was formed with longitudinal timber bulkheads, which were of 6 by 8-in. posts about 10 ft., center to center, sheathed with 2-in. planks for a height of 5 ft. The embankment sides were thus built up in steps 4 ft. high and 8 ft. wide. The posts were tied back to stakes fronted by planks. The pipe extended longitudinally along the center line of the embankment from the river so as to bring the embankment up to final grade and working away from the dredge. It discharged on a frame of baffle-boards spreading the water and depositing the sand. The bulkhead planks were removed and re-used and the embankment sides finished to an even 2 to 1 slope. The embankments were finished by fertilizing and planting the sides with grass and indigenous shrubs, including about 4 000 Scotch broom plants and 2 000 rose-bushes. The pavements are bitulithic on a broken-stone base.

At each end of the main river bridge there are two entrance columns or pylons of concrete, together with short lengths of ornamental concrete hand-rail. At the Vancouver end a small drinking fountain is included. These pylons bear bronze tablets giving the names of those associated with the work, with summaries of quantities and costs, and also inscriptions representing something of the purpose and ideals of those responsible for the bridge.

Costs.

The total cost of the entire structure and of the principal separate parts follows:

Columbia River Bridge	$891 005.45
Oregon Slough Bridge	196 944.47
Columbia Slough Bridge	74 942.99
Vancouver Approaches	16 086.87
Union Avenue Approach	251 220.26
Derby Street Approach	104 877.76
Lighting system	16 014.05
Ferry landings	16 890.24
Tracks on Vancouver streets	7 139.99
Right of way	25 449.63
Fees to engineers	80 000.00
Miscellaneous	2 984.61
Total	$1 683 556.32

In Table 2 the principal unit prices paid to contractors are listed.

TABLE 2.—Principal Unit Prices Paid to Contractors.

Material.	Main river bridges.	Slough bridges.
Concrete in pier shafts, per cubic yard	$9.00	$10.00
Mass concrete in bases of piers, per cubic yard	12.00	11.00
Piles below pier bases, per linear foot	0.45	0.35
Rip-rap around piers, per cubic yard (solid)	2.05	2.00
Metal in truss and girder spans:		
Manufacture and delivery, per pound	0.0259	0.0255
Erection and painting, per pound	0.0055	0.0055
Metal in towers:		
Manufacture and delivery, per pound	0.0343	.....
Erection and painting, per pound	0.0090	.....
Machinery and ropes:		
Manufacture and delivery, per pound	0.10	.....
Erection, per pound	0.0125	.....
Electric railway rails and fastenings:		
Manufacture and delivery, per pound	0.02755	.....
Erection, per pound	0.0040	.....
Concrete in counterweights, per cubic yard	14.00	.....
Concrete roadway, slabs, and bitulithic paving, per square yard	3.48	3.48
Hayden Island and Union Avenue Approach:		
Embankment in place, per cubic yard	0.1324	.....
Broken stone paving base, per cubic yard (loose)	1.48	.....
Bitulithic paving on broken stone base, per square yard	1.17	.....
Concrete slope protection slabs, per square yard	0.80	.....
Derby Street Approach embankment in place, per cubic yard	0.1648	.....

PART III.—A BRIDGE WITH A LIFTING DECK.

The North Kansas City Bridge over the Missouri River.

The Union Depot and Terminal Railway Company's bridge over the Missouri River at Kansas City, Mo., called the North Kansas City Bridge, was completed in 1912 at a cost of $2 400 000. It is a double-deck structure, Fig. 4. The lower deck extends across the river only, a distance of 1 280 ft., and carries a standard double-

track railway. The upper deck provides double track for electric cars in a central roadway which also serves fast motor vehicles, two side roadways, and two sidewalks from the south end of the bridge to the north side of the river, where stairs give access to the ground. The length between the ends of the upper-deck approaches is 5 340 ft.

The main, or river, portion of the bridge consists of three heavy riveted truss spans each 428 ft. long. Two of these spans carry the highway deck at the top chord and the railway deck at the bottom chord. The third span is placed at a higher elevation so that its lower chords are just below the upper deck. The railway deck of this span is suspended from the main trusses and can be raised to permit the passage of boats. The navigable channel is 412 ft. wide between piers and, with lower deck raised, has a vertical clearance of 65 ft. above standard high water.

The southerly approach to the river bridge is 1 630 ft. long, consisting of 1 156 ft. of steel viaduct and spans and 474 ft. of earth embankment. The northerly approach is 2 428 ft. long, with 1 708 ft. of steel viaduct and spans, and 720 ft. of embankments. The railway tracks on the lower deck of the river spans curve out under the approach viaducts at each side of the river.

Approximate Quantities.

The following tabulation of the quantities for the North Kansas City Bridge shows the approximate total superstructure metal, piling, and substructure masonry required:

Superstructure Metal:		
South approach viaduct	885	tons
South approach spans	1 617	"
North approach viaduct	2 460	"
North approach spans	236	"
Two fixed spans	8 037	"
Lift span:		
Carbon steel	4 203	"
Nickel steel	346	"
Machinery	292	"
Total superstructure metal	18 076	tons
Side roadways	13 500	sq. yd.
Sidewalk	2 100	" "
Counterweights	380	cu. yd.
Substructure masonry, total	29 000	" "
Timber piles, 336	13 500	lin. ft.
Concrete piles (Chenoweth), 1 395	38 600	" "
Creosoted timber	950 000	ft. B. M.
Untreated timber	250 000	" "

Confronting Conditions.

In the early Nineties a high-level, single-deck, railway and highway bridge was projected at this site and the piers were built, but lack of funds suspended further construction. The river was wider than at present, and the bridge was to include four main spans of 428 ft., and approaches. The property finally came into the possession of the present owners as part of large holdings on both sides of the river. Harbor lines were established by the Government, and extensive reclamation work confined the river to about three-fourths of its former width. Plans for the present bridge were started in 1907.

The new conditions required not only a high-level deck, with approaches over the tracks on both sides of the river, to accommodate highway traffic and street and interurban cars from St. Joseph, Excelsior Springs, and other points north of the river, but also a double-track railway deck at the level of the river valley to connect the terminal holdings of the Company. This made necessary some kind of movable span, because a fixed bridge which would provide for the lower level tracks would not afford vertical clearance for boats. The original piers were well built of excellent limestone masonry laid in cement, and were in good condition. They rest on bed-rock 50 to 60 ft. below low water. Originally built with starlings, they are 46 ft. long at the water level, thus sufficing for the width of a double-track bridge.

To construct a pivot pier between two of the existing piers for an ordinary swing span would limit the channel openings to about 160 ft., which is less than has been required for any swing span built in recent years on this river. To construct a swing span with two 200-ft. channels involved not only a pivot pier and draw protection, but also one new end pier, and the removal of an existing pier. The resulting short span in the river would have been uneconomical and somewhat unsightly. A bascule for such a wide opening could not be considered. Other possibilities were the lifting span or lifting deck. The lifting deck span offered a fixed upper roadway with uninterrupted highway and street-car travel, and involved only slight changes in the existing substructure. It appeared to be nearly $200 000 less costly than a bridge with a swing span of the necessary length. The lifting deck idea also was adapted without material alteration to an uncertainty as to the ultimate future elevation of the lower deck railway tracks. Although for the immediate development a track elevation 24 ft. above high water was suitable, there was a possibility that an elevation perhaps 13 ft. higher might be necessary.

Loads and Working Stresses.

The live loads and impact assumed in the design were: For railways, Cooper's Class E-50; for street railways, a continuous line of 50-ton cars and 30-ton special trucks; for roadways, a 15-ton road roller or truck, and varying floor loads. Percentages for impact were added in all cases. The total live and impact loads on two trusses of a 423-ft. span, were 14 100 lb. per lin. ft.

The unit stresses used in the design correspond closely to those specified by the American Railway Engineering Association, and are as follows: For nickel steel 20 000 lb. per sq. in., in tension, with other working stresses to correspond; for reinforced concrete in compression 650 lb. per sq. in. and for tension in steel 15 000 lb. per sq. in.; load per pile, 27 tons; load on bed-rock 8.6 tons per sq. ft. maximum.

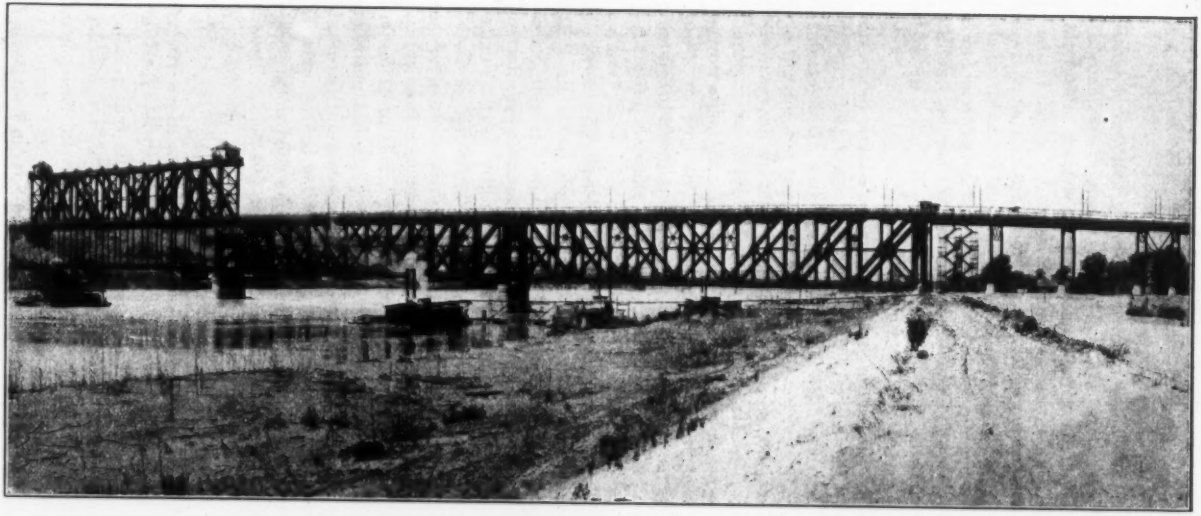


FIG. 4.—GENERAL VIEW OF NORTH KANSAS CITY BRIDGE FROM DOWN STREAM.

Substructure.

The four existing river piers were cut down to below the old starlings, and were capped with concrete copings 8 ft. thick. The other old piers were used almost without alteration, their shortness being compensated for by cross-girders long enough to extend out to the truss shoes of the span supported.

The approach viaducts rest on concrete pedestals supported on concrete piles, and in part on timber piles. The concrete piles were 33 ft. long, pre-cast, of the Chenoweth type, and were jetted and driven. The cost of remodeling the old substructure and building the new one was about $135 000. The old piers cost when built about $600 000.

Superstructure.

The river spans are of riveted construction; two are exactly alike and the third is somewhat similar. The trusses, which are 32-ft. centers transversely, have parallel chords, sub-diagonals, and hangers. The floors are of the usual character with wooden cross-ties for the tracks; for the upper roadway the ties are double-planked. Cantilevers beyond the trusses, at the upper deck, support at each side, on longitudinal stringers, a reinforced concrete slab, comprising a roadway 13 ft. 6 in. wide and a narrow sidewalk. These side roadways are paved with asphalt.

The chords are 5 ft. deep and the top chord cover-plate is 5 ft. wide. The maximum gross section of chord and end post is about 500 sq. in. Gusset-plates are spliced into web-plates of members, with rivets in double shear wherever possible, thus making the gusset-plates of minimum size and limiting the secondary stresses in the members. Even with these shingled gusset-plates the main members were more than 16 ft. high above the rails when loaded for shipment, and closely approached the maximum size which could be shipped. The lacing on the open sides is of substantial angles connected by four rivets at each end to small lateral plates, with the angles turned within the member. The mid-sections of the chords and the end posts are each about 100 ft. long, and weigh from 96 to 106 tons each.

The clearance for gusset-plates was $\frac{1}{8}$ in., requiring nicety of shop work and ability in erection to insert members of such size and weight between gusset-plates for distances of from 12 to 16 ft. Where the grip exceeded 5 in., the rivets were coned to make upsetting easier.

Instead of the usual pin-plates and shoe-pin at the ends of the trusses, cast-steel diaphragms were riveted between the gusset-plates. The lower end of the casting and the shoe below both afford full-length bearing for a 12-in. pin, which carries a total reaction of 2 400 tons, but is not subjected either to shear or bending.

Secondary Stresses.

The secondary stresses in main truss members were calculated and means used to reduce them to the lowest practicable limit. In general, the truss members were fabricated of lengths so that all would be straight and the chords horizontal when the whole truss carries full dead and one-half live and impact loads. The main diagonals and main posts, however, are straight under their own maximum conditions of similar loading. To reduce the secondary stresses due to chord stiffness, the chords were assembled on the ground in straight lines and the field splices reamed. When erected on the falsework the chords were again assembled in straight lines and the splices fitted up with full-sized bolts and pins. The chords were

then jacked into cambered position for no load and the web members connected and riveted. Under loads which bring the chords down to horizontal, the secondary stresses are therefore a minimum.

By an arrangement with Dr. W. S. Stratton, Director of the U. S. Bureau of Standards, an effort was made to obtain direct measurement of actual stress distribution. Before erection, fixed points were set about 16 in. apart on several members, and micrometer gauge readings were made of the exact lengths, with due regard for temperature and other conditions. Corresponding readings were made after erection and the comparison gave some indication of the amount and distribution of stresses, but the work has not been fully completed or classified for use.

Lifting Deck Span.

The lifting deck span is supported on two bents, Fig. 5, about 60 ft. high, with lateral bracing between columns. The columns are fixed transversely at their lower ends by anchor-bolts which connect to heavy girders embedded in the piers. The supporting bent at one end of the span is part of a viaduct tower and provides a fixed end. The bent at the expansion end is a rocker pivoted on pins above and below.

The lifting deck of this span is an ordinary double-track railway bridge deck with four lines of continuous stringers riveted into cross floor-beams, with a lower lateral system and bottom chords. Each floor-beam is suspended from the truss above by two stiff member hangers that rise into the truss posts above them when the deck is lifted. When in position for train traffic, the ends of the lifting deck rest in shoes on the piers, and all wind and vibration stresses are carried through the lateral system directly to the masonry. The vertical live loads on the lower deck are transmitted to the trusses of the supporting span through pins in the tops of the hangers which seat in sockets on pairs of diaphragms in the truss posts.

The dead load on each hanger is taken by two 1½-in. wire ropes which pass up through the truss posts, over an idler sheave on the top chord, thence to the nearest adjacent corner of the span, where they pass over a grooved drum which accommodates one-fourth of all the suspending ropes, thence downward, and connect to a concrete counterweight. At the ends of the lifting deck no hangers are used, but the ropes extend down and connect to the end floor-beams. The corner drums are so grooved and so placed that the suspending ropes above the top chords diverge at slight angles so that each pair just clears the sheave at the next panel point. Leaving these corner drums the ropes again fan out so that they meet a row of deflecting sheaves at the upper deck level in a line 3 ft. 7 in. outside the center line of the trusses. The side lead of the ropes does not exceed ¼ in. per ft.

Each floor-beam dead load is thus balanced by two counterweights. These counterweights hang, in four groups, in rows just outside the planes of the trusses, on both sides. Each counterweight has small jaw guides which engage on vertical tracks of Z-bars attached to the tower bracing or truss members. The lifting deck is thus counterbalanced by thirty-two counterweights of separate concrete blocks.

Operation by Corner Drums.

Operation is effected by rotating the upper corner drums, for the friction of the suspending ropes on them is sufficient to raise and lower the deck. The drums at each end are on a common shaft and so connect through a train of gears to an electric motor. Mechanical synchronization of machinery at opposite ends, insuring consequent uniform movement of all points of the lifting deck, is secured by two counterweighted rope drives connecting opposite motor shafts. One rope drive has the taut line for driving in one direction, and the other has the taut line for the reverse. The motors are also connected electrically, and under ordinary conditions both operate simultaneously. In case either should be out of service the lifting deck can be operated by a single motor from either end, the machinery at the opposite end being operated through the rope drive.

The lifting deck is guided in its movement by jaw castings at its four corners, attached to the lower chords, which move astride angles on the main columns. Near their lower ends these vertical guides flare out slightly and bring the deck to exact lateral position as it approaches its seat. No other guides are used. The hangers are so much smaller than the truss posts that with maximum deflection they cannot come into contact, and they hang wholly free.

When the lifting deck is down ready for train service it is locked in position by end locks on the piers and by locks at the top of each hanger. All these locks are drawn simultaneously at will, but each automatically and independently locks as the lifting deck comes to its seat, each lock having an operating counterweight. The function of the hanger locks is to overcome the slight lifting tendency of the continuous stringers under an advancing load. They consist of pairs of cams swinging on shafts in the posts so as to bear against seat castings on the hangers.

Motor-operated machinery, located in one of the machinery houses, draws the locks. It lifts all the counterweights over their neutral points so that each may be tripped and will then fall and throw its lock. All locks have electric contacts causing signal lamps to glow when the locks are withdrawn.

The lifting deck is operated by two railway type motors in series-parallel, equipped with solenoid brakes. Magnet type controllers are used. Limit switches in emergency cut off the current and apply the brakes near each limit of movement. An indicator in the house shows the position of the lifting deck.

To save weight, the lifting deck and hangers are of nickel steel. The total load lifted is 800 tons, and the total moving load is about 1 700 tons. The maximum movement is 50 ft., which is traversed in less than 1 min.

There is much duplication in machinery parts. Practically all deflecting sheaves are of one pattern. The operating drums are alike and the operating machinery at the ends is in duplicate. A concrete and steel house encloses the machinery at each end.

Manufacture and Erection.

Except for the 346 tons of nickel steel in the lifting deck and hangers, the bridge is of medium, open-hearth, carbon steel, of substantial Manufacturers' Standard Structural grade. The nickel steel has an ultimate strength of from 85 000 to 100 000 lb., a minimum elastic limit of 50 000 lb., and is 3.25% nickel.

The shop work was excellent. So accurately were abutting ends of chords milled that the blade of a pen knife could find no entrance around the entire perimeter of these members 5 ft. square. All rivet holes were sub-punched and reamed or drilled. Those for important field connections were reamed or drilled with the members assembled. This reaming and drilling of field holes for the main trusses was done near the bridge site where yard space nearly ½ mile long was used for laying out the trusses.

The three river spans were erected on pile falsework with an overhead steel traveler equipped with electric hoists. For the lifting deck span, framed bents in two lifts were built on the pile bents, so that the total height of the falsework above the river bed was about 120 ft. The larger members were lifted with special clamps which gave direct connection to the pins of the blocks.

The lifting deck was erected in raised position about 12 ft. below its upper limit, supported on temporary pins in special pin holes in the hangers. The concrete counterweights were built on the ground or on stagings below their respective positions, and after the suspending ropes had been laid in place and connected to the deck, were raised and attached to the ropes. The lifting deck was then raised slightly by the operating machinery and the temporary pins were removed.

Personnel and Prices.

The foundations, as originally constructed, were designed by Mr. Frank B. Moore. The new work was designed and construction supervised by the writer's then firm, Waddell and Harrington. The principal work, that of manufacture and erection of the superstructure, was in the hands of the McClintic-Marshall Construction Company. The late Paul L. Wolfel, M. Am. Soc. C. E., Chief Engineer of that Company, made many valuable suggestions concerning the design, manufacture, and erection of the large riveted spans. Much credit also is due to Mr. F. W. Fratt, an engineer of wide experience, then President of the Union Depot Bridge and Terminal Company. Work began in 1909 and the bridge was opened to traffic early in 1912.

The principal prices per pound for the superstructure were: Carbon steel for the viaduct on tangent, delivered 2.55 cents, erection 0.5 cent per lb.; for the viaduct on curve, correspondingly, 2.6 and 0.5 cents; for the main river spans, correspondingly, 2.76 and 1.125 cents; for nickel steel, delivered, 4.7 cents, erection 1.185 cents; for machinery, correspondingly, 6.9 and 1.25 cents; for wire rope, 14 cents. The total cost of providing the lifting deck was $125 000 more than the cost of a bridge of fixed simple spans throughout.

Model Demonstration Prior to Construction.

In order to demonstrate to certain of the owners who could not wholly visualize the methods of operation from the drawings, a complete model of the movable span was built (Fig. 6). It was made to a scale of 1:15, so that the span length was 28 ft. 7 in., and the distance between truss centers was 25½ in. The members were of wood about 4 in. square, fastened together with steel gusset-plates. Machinery was built conforming in every essential part to that proposed for the bridge, and electric motors and drives were installed and connected in the same manner with duplicate rope drives. The lifting deck was made to weigh one one-thousandth of the

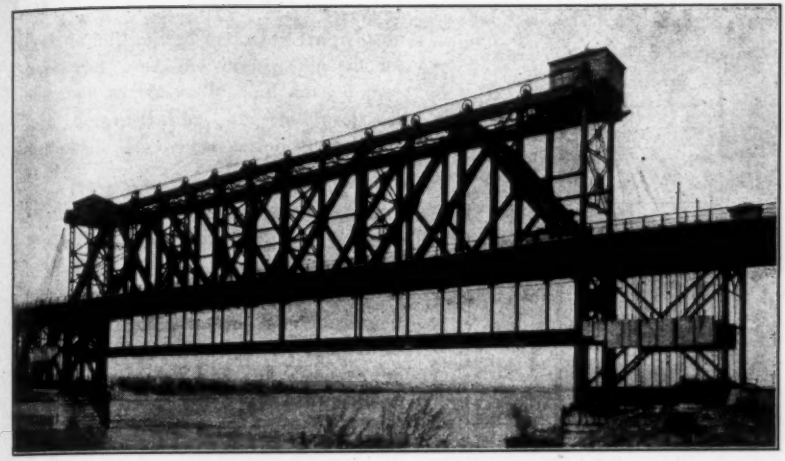


FIG. 5.—NORTH KANSAS CITY BRIDGE: LIFTING DECK PARTLY RAISED.

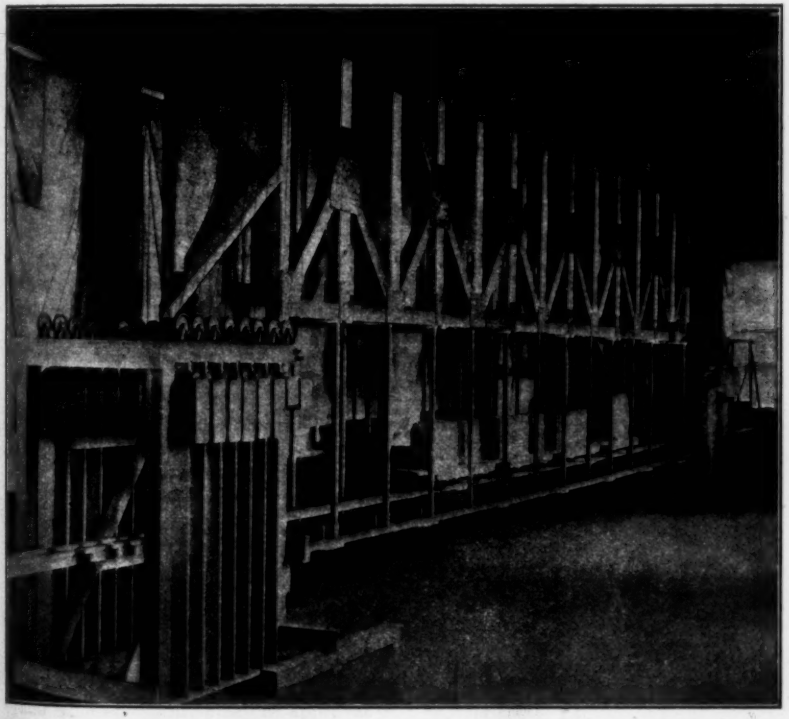


FIG. 6.—MODEL OF LIFT SPAN, NORTH KANSAS CITY BRIDGE, SHOWING DECK DOWN AND LOAD ON DECK.

deck as built. Proportionate train loads were run over the deck. The model operated satisfactorily, and gave a perfect demonstration of the proposed methods. Its construction cost was $1 500.

Rope-Drive Experiments.

To meet criticisms of the proposed rope drives made by certain manufacturers of rope-driving machinery, a physical demonstration was arranged (Fig. 7). Driving machinery similar to that on the bridge, but with a sheave for a single suspending rope instead of drums, was placed on two platforms, 320 ft. apart. Each driver was operated by a 50-h. p. motor, which was also partly loaded by a brake. The two drivers were connected by a double rope drive. A loaded platform 24 ft. long was suspended midway between drivers, within a framework, by suspending ropes which extended over the sheaves on the drivers and connected to two counterweights composed of two cast-iron pile hammers. This suspended span was raised and lowered through 16 ft. without the slightest deviation of movement, as between ends, when operated by both motors or by either motor from either end, demonstrating the synchronism afforded by the rope drives.

A committee of engineers, consisting of Thomas E. Brown, M. Am. Soc. C. E., Professor W. F. M. Goss, of the University of Illinois, S. B. Fisher, M. Am. Soc. C. E., then Chief Engineer of the Missouri, Kansas and Texas Railway Company, and Mr. George W. Jackson, of Chicago, Ill., was appointed to consider the proposed method of operation. This committee examined the preliminary designs of the machinery, inspected the model and the experimental rope drive, and reported unanimously in approval of the proposed construction.

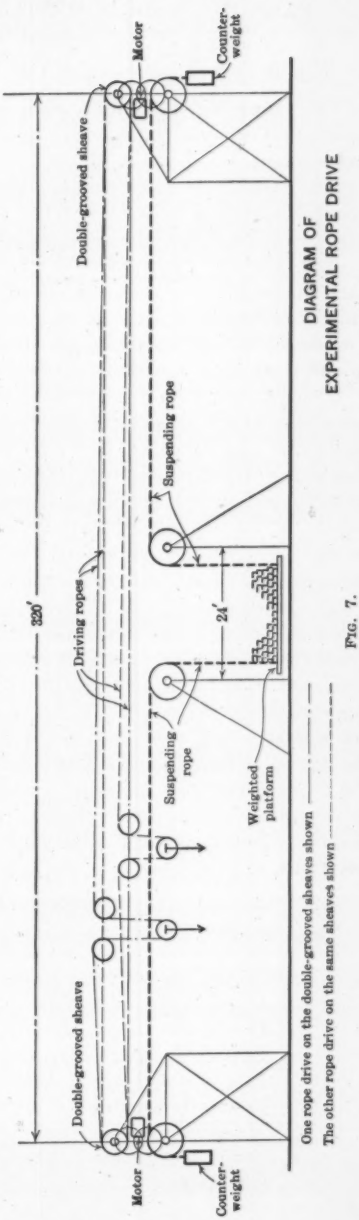


FIG. 7.

PART IV.—A BRIDGE WITH A LIFT SPAN WHICH HAS A LIFTING DECK.

The Harriman Bridge over the Willamette River at Portland, Ore.

The Oregon-Washington Railroad and Navigation Company's Willamette River Bridge at Portland, Ore., completed in 1912 at a cost of $1 715 000, is a double-deck structure which is 1 900 ft. long between the ends of the upper-deck approaches, Fig. 8. The lower deck extends across the river only and, including short approach spans on one side, is 900 ft. long between abutments. The upper highway deck, 73 ft. wide, accommodates double street-car tracks, and has a paved roadway and two sidewalks; the lower deck provides for a standard double-track railway. The bridge is in the business district of Portland near the Union Station. The railroad tracks enter the deck from the terminal yards with a curve of 15° and pass from the bridge on the opposite side of the river, both up and down stream, with 16° curves.

Of the three riveted truss spans over the river, the two side spans are fixed, and the center span is movable and is suspended between two towers supported on the fixed spans. To accommodate the track alignment, both fixed spans are of the spread type with non-parallel trusses. The westerly fixed trusses, 287 ft. long, are 34 ft. at the river end and 40 ft. 3 in. at the shore end between centers; the easterly fixed trusses, of the same length, are correspondingly 34 ft. and 71 ft. 6 in. between centers. The fixed spans have the railway floor near their lower chords and the highway deck above their top chords.

The movable span is a through truss lift span for the highway floor, with a suspended lifting deck for the railway floor, Figs. 9 and 10. For the passage of a river steamboat or tug this lifting deck can be raised to contact with the upper floor, its supporting hangers telescoping into the posts of the upper trusses, affording vertical clearance of 72 ft. above ordinary low water. Such movement does not interrupt traffic on the upper deck. For the passage of vessels with masts the lift span, including the suspended lifting deck, may be raised to a height of 140 ft. above extreme high water, or 173 ft. above low water. The lifting deck may be fully raised or lowered in 30 sec., and the lift span in 60 sec. Operation is by electric power; the total moving load with the lifting span in operation is about 4 600 tons.

Approximate Quantities.

The approximate quantities in the structure are:

Superstructure Metal:

Upper deck approaches	1 209 tons
Lower deck approaches	103 "
Fixed truss spans (2)	4 262 "
Lift span and lifting deck	1 600 "
Towers supporting lift span	871 "
Machinery on span and towers	559 "
Street railway track material	125 "
Gas pipe	51 "
Total	8 780 tons

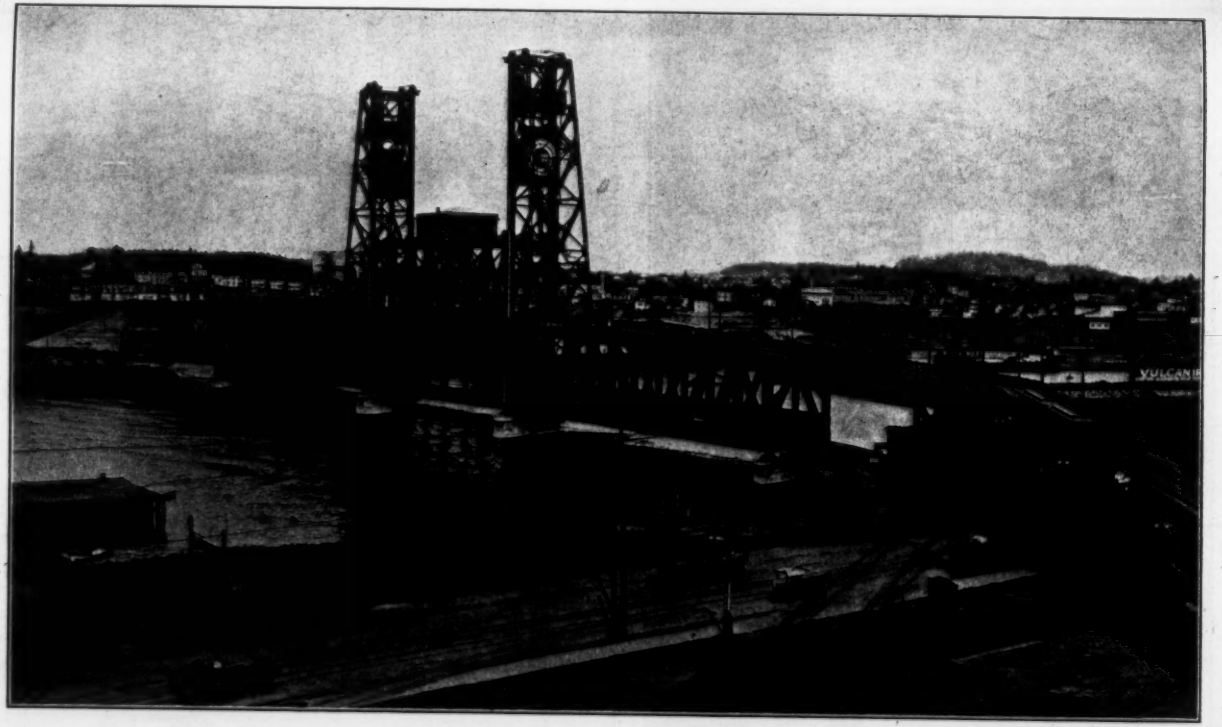


FIG. 8.—HARRIMAN BRIDGE: VIEW OF COMPLETED STRUCTURE LOOKING EAST, SHOWING ENTRANCE OF RAILWAY TRACKS AT WEST END, AND RELATION OF HIGHWAY APPROACH.

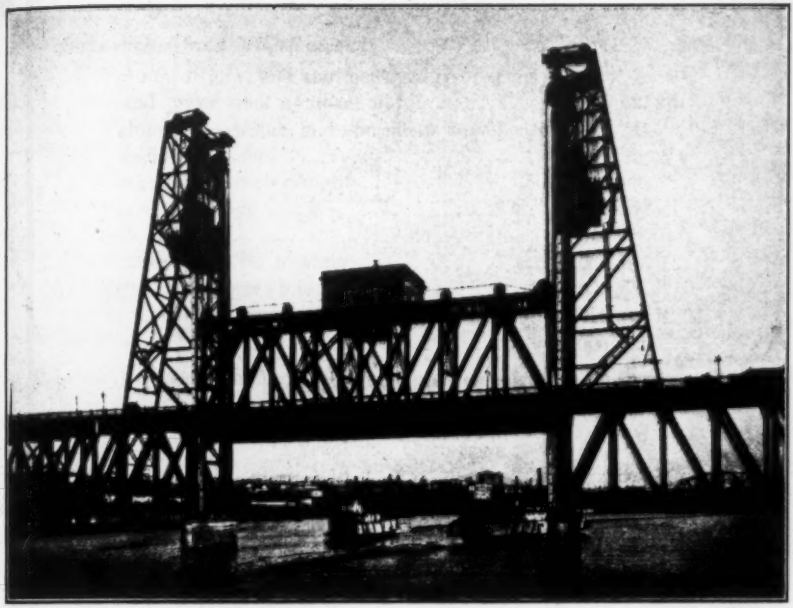


FIG. 9.—HARRIMAN BRIDGE: VIEW SHOWING LIFTING DECK RAISED; RIVER STEAMBOATS PASSING BENEATH.

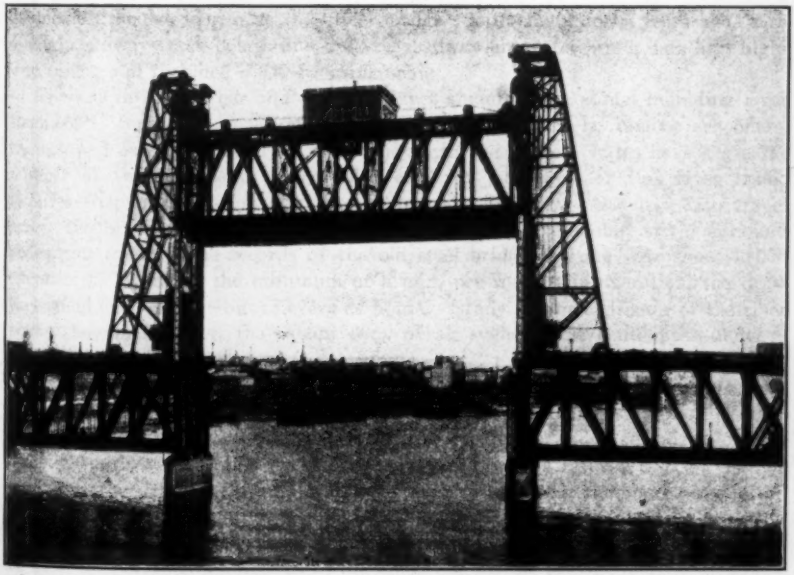


FIG. 10.—HARRIMAN BRIDGE: VIEW SHOWING LIFTING SPAN AND LIFTING DECK RAISED TO EXTREME HEIGHT.

Substructure and Miscellaneous:

Volume of piers and abutments on river spans and lower deck approaches	23 300	cu. yd.
Volume of pedestals and abutments for upper deck approaches	6 700	" "
Concrete in counterweights	1 080	" "
Paved roadways	10 300	sq. yd.
Sidewalks	2 600	" "
Timber piles 704, totaling	35 000	lin. ft.
Raymond concrete piles 222, totaling	7 875	" "
Timber in superstructure	700 000	ft. B. M.
Cement	38 000	bbl.

Construction began in May, 1910, and was completed in August, 1912. The movable span is absolutely unique—a logical solution of the problem offered.

CONFRONTING CONDITIONS.

The Willamette River flows northward through the City of Portland and discharges into the Columbia River some 8 miles distant. The principal business district of the city and a large residence section are west of the river, and to the east the largest residence section and some business and industrial enterprises are located. This produces an enormous trans-river highway traffic over the five existing bridges, each of which has a movable span. The Oregon-Washington Railroad and Navigation Company's tracks enter the city from the east and the Southern Pacific tracks from the south, and all trains must cross the river to reach the station and terminals on the west side. These were formerly served by a double-deck bridge known as the "steel bridge", built about forty years ago, with a single railway track below, and a 32-ft. roadway above for street cars and highway traffic, and included a 340-ft. swing span.

There is in the Harbor of Portland a large amount of shipping, including river steamboats, tugs, and small craft, ships and sailing vessels, lumber schooners, tramps, and ocean liners. Few river harbors in the United States have a greater number of vessels passing daily. The constant movement of this river traffic requires frequent operation of the movable spans, interferes seriously with travel across the bridges, and causes repeated congestions of vehicular, street car, and pedestrian traffic. The records of the old steel bridge for one year show 35 000 operations. Allowing the minimum of 5 min. per round-trip operation, the draw thus stood open about 8 out of every 24 hours. Study of the character of the river traffic showed, however, that about 80% of all such vessels could pass under a bridge 60 ft. high.

These conditions confronted the officials of the Railway Company when, because of deterioration of the old structure and the need of double-track facilities for heavy loading, the question of a new bridge was taken under consideration. A combined railway and highway bridge was desired, with a railway deck at the low level of the terminal yards, and a highway deck so high that the approaches would pass over the adjacent yard tracks. It was evident that there would be interruption to traffic on the highway deck only a few times daily if the lower deck could be made to operate independently of the upper one for the passage of river steamers.

To meet these conditions the writer proposed the double-action lift span. Preliminary plans and estimates were submitted and although no such structure had ever been built, the railway officials were convinced that no problems would be involved not soluble along rational, well established practice. In addition to advantages of operation, comparative estimates of cost showed that the bridge with the double-action lift would cost $250 000 less than a bridge including an ordinary double-deck swing span.

Loading and Specifications for Design.

The Harriman Lines' Common Standard Specifications Nos. 1006 and 1012 for railway and highway bridges, with slight modifications and supplemented by special requirements, were used for design. The assumed live loads were as follows: For the railway, two consolidation locomotives, Class E-55, followed by a uniform load of 5 000 lb. per lin. ft.; for the highway, a 24-ton truck in a space 12 by 20 ft., and 100 lb. per sq. ft. over the remaining area; and for street-car tracks, two 50-ton cars coupled, with a minimum of 1 200 lb. per lin. ft. per track. The allowable working stresses of the specifications are somewhat less than those permitted in the current American Railway Engineering Association Specifications for Railroad Bridges.

Substructure.

The river at the site is 800 ft. wide, with a maximum depth of 90 ft., and exceeding 40 ft. over most of its width. Hard conglomerate—"cemented gravel"—exposed at the water's edge on the east bank, is 125 ft. below low water at mid-river, and quite below reach at the west bank. The east side afforded good foundations for approach pedestals, but piles were needed on the west side, as much of the area there is filled ground.

The two main piers are founded on hard cemented gravel, one 120 ft. and the other 130 ft. below low water. These were constructed by the open dredging process. Their bases are 36 ft. wide and 72 ft. long. Each is a timber crib filled with concrete. The crib contained six wells for dredging, each 9 by 11 ft., from 11 ft. above the cutting edge, upward. Below that point the wells flared out and occupied the whole area of crib at its bottom. The entire perimeter of the crib, its central longitudinal wall, and two cross-walls, were shod with heavy steel cutting edges. The crib walls, generally 1 ft. thick, had the spaces between them filled with concrete, as the cribs were settled to position, thus making walls of timber and concrete, 5 to 6 ft. thick, around the wells. After being duly founded on the bed-rock, these wells were filled with concrete deposited under water up to a point where it was convenient to pump out and place the concrete in the open. The tops of the cribs are about 20 ft. below water, where the neat work of the shafts begins. The shafts were constructed within removable timber coffer-dams bolted to the cribs.

Excavation for Pier Base.

A substantial dock of about 100 piles, braced and capped, was first built for each pier. Borings were made over the area to be covered by the pier, and the elevations of hard material determined. For both piers this bed-rock was found to be on a very considerable slope. In Pier No. 4 the slope amounted to 19 ft. between diagonally opposite corners (Fig. 11). Pipes 4 in. in diameter were driven

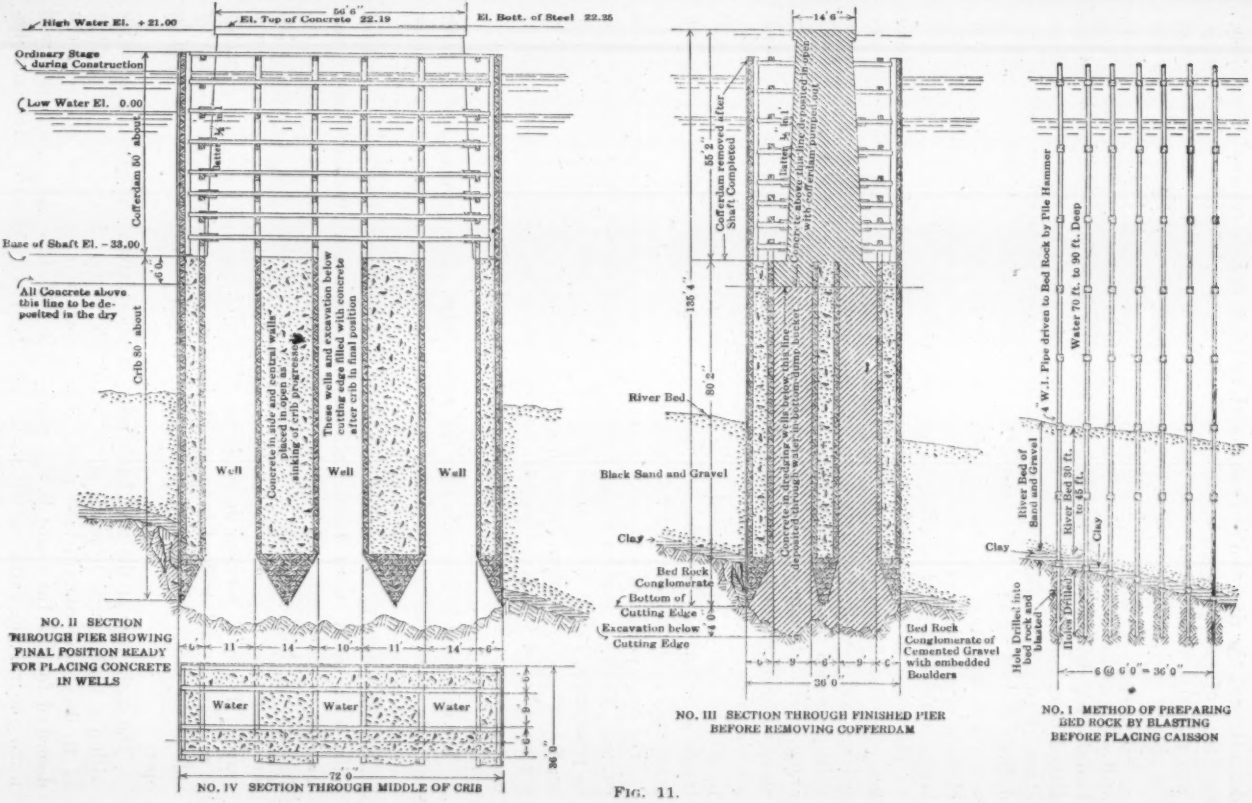

High Water El. +21.00
El. Top of Concrete 22.19
El. Bott. of Steel 22.35
Ordinary Stage during Construction
Low Water El. 0.00
Cofferdam 60' about
Base of Shaft El. -33.00
All Concrete above this line to be deposited in the dry
Crib 80' about
Well
Well
Well
Concrete in side and central walls deposited through water as sinking of crib progresses
These wells and excavation below cutting edge filled with concrete after crib in final position
Water
Water
Water
11
14
10
11
14
6
72' 0"
NO. IV SECTION THROUGH MIDDLE OF CRIB
NO. II SECTION THROUGH PIER SHOWING FINAL POSITION READY FOR PLACING CONCRETE IN WELLS
14' 6"
Cofferdam removed after Shaft Completed
River Bed
Black Sand and Gravel
Clay
Bed Rock Conglomerate
Bottom of Cutting Edge
Excavation below Cutting Edge
Concrete in deepening wells below this line deposited through water in bottom dump bucket
Concrete above this line deposited in open with cofferdam pumped out
Bed Rock Conglomerate of Cemented Gravel with embedded Boulders
36' 0"
NO. III SECTION THROUGH FINISHED PIER BEFORE REMOVING COFFERDAM
4" W.I. Pipe driven to Bed Rock by Pile Hammer
Water 70 ft. to 90 ft. Deep
River Bed of Sand and Gravel
River Bed 30 ft. to 45 ft.
Clay
Clay
Holes Drilled into Bed Rock and Blasted
Holes Drilled
6 @ 6' 0" = 36' 0"
NO. I METHOD OF PREPARING BED ROCK BY BLASTING BEFORE PLACING CAISSON

FIG. 11.

down through the river bed of sand, gravel, and clay to hard material, successively, over the entire area of the pier at intervals of 6 ft., and holes were drilled into the hard material to a selected elevation. The contained material was washed out of the pipe by a jet, and a 4-in. drill bit on a 2-in. pipe was used inside the larger pipe to drill into the hard material. When 5 or 6 ft. of the hole had been drilled, the drill was withdrawn, and a sheet-iron tube, 2½ in. in diameter and about 5 ft. long, loaded with dynamite and powder, was carefully lowered to the bottom of the drilled hole, and weighted. The casing pipe was drawn up about 10 ft. and the charge fired. The casing pipe was then driven down as far as possible, and the drilling started again. This process was repeated at intervals deemed sufficient to produce a bottom uniform in character over the area of the pier base. The blasting of the bed-rock material was thus carried on before any excavation was begun in the 40 to 70 ft. of overlying river-bed material.

The lower 30 ft. of the pier crib was built on shipways, on shore. It was launched and floated to a position within the temporary dock which was then finished and closed. Concreting in the wall spaces and building up the timber walls proceeded simultaneously. The dredging for sinking the pier was done with orange-peel and clam-shell buckets, and with a hydraulic jack, or water siphon, operated by large pumps. In each pier the area blasted out proved to be too small, for when nearing final position the pier hung on the sides of the excavation. Additional blasting in holes drilled 4 ft. outside the cutting edge blew the material inside so it could be removed, and the crib finally reached a satisfactory position practically at the elevation predetermined.

The total height of each pier is about 145 ft. and each contains about 8 650 cu. yd. The total load on a pier foundation, including live, dead, and pier load, is practically 16 000 tons. Ignoring side friction, the intensity is 6 tons per sq. ft. The side friction developed during sinking was calculated to be 550 lb. per sq. ft., and with this friction the net load on the foundation would be 3.7 tons per sq. ft. The river has practically no ice, and the pier shafts are of concrete without special protection.

The east main abutment is near the water's edge. The extreme depth of water was only 25 ft., but conditions for coffer-dam construction were difficult because the cemented gravel bed-rock, too hard to be penetrated by a steel-shod wooden pile, was overlaid only by a few feet of loose, round boulders. Two rows of old steel rails, 5 ft. apart, were driven into the hard material. Waling timbers were fastened to these rails and triple-lap wooden sheet-piling was placed within and driven to the hard material. A rich grout mortar was deposited in the coffer-dam through a pipe, to cement the boulders together and make a seal to the cemented gravel. The space between the walls was filled with clay and loam, and a quantity of mud and gravel dredged from one of the river piers was piled against the river side of the wall. The coffer-dam was then pumped out and work proceeded in the open.

The main pier at the west bank rests on piles driven and sunk by water-jets within a timber crib, to about 90 ft. below low water. The crib is 33 by 69 by 25 ft. deep, and contains 233 piles. The construction was very similar to that described for the Interstate Bridge piers. The pedestals and abutments were of usual construction, partly on soil, partly on timber piles, and partly on concrete piles.

Superstructure.

Over the three river spans the upper deck is 73 ft. wide, with a central roadway of 27½ ft., two side roadways of 15 ft., and two 6-ft. sidewalks. The approaches are, in general, 55 ft. wide, with one 42-ft. roadway and two 6-ft. walks. The grades and alignment of the upper deck approaches were restricted by physical conditions, and somewhat by civil authorities. The west approach has a grade of nearly 7%, and the east approach a grade of 2.5 per cent.

These approaches are steel viaducts of the usual type. The floor is of 4-in. creosoted plank on creosoted fir ties, with creosoted wood block pavement. The hand-rails are of heavy steel design. The sidewalks are of plank covered with an asphaltic preparation ¼ in. thick. The street-car rails are 8 in. high.

Towers and Machinery.

The towers (Fig. 12), from which the lift span and the lifting deck are suspended, are integral parts of the fixed spans. The main posts, which support all the weight of the lift span and its counterweights and about three-fourths of the weight of the lifting deck and its counterweights, rest on the shoes which also serve the fixed spans. The rear posts of the towers connect at the second panel points of the spans, and support about one-fourth of the weight of the lifting deck and counterweights. The main sheaves which support the lift span rest on bearings built up from trusses connecting the front tower posts. The sheaves for the lifting deck rest on girders at the tops of the towers. The towers are so wide that both counterweights can move within them. The main counterweights are suspended just back of the front posts. The lifting deck counterweights are back of the main counterweights, requiring double sets of sheaves.

The towers are designed for a wind pressure of 15 lb. per sq. ft. with span moving, and 30 lb. with the lift span down. The total load on each main post shoe is 3 250 tons, which is supported on a cast-steel base 12 by 12½ ft. The load is carried into the shoe by a pin, 13¼ in. in diameter, in bearing over its full length. A steel casting riveted between the gusset-plates and supporting the tower post affords bearing on the pin.

The lifting deck is a usual double-track railway deck of stringers and floor-beams with a lower lateral system and bottom chords. It is suspended from the trusses of the lift span by stiff member hangers which rise centrally inside the truss posts when the deck is lifted. The end shoes on the piers, the supporting pins and diaphragms, end locks and hanger locks, and other details are similar to those of the North Kansas City Bridge. Each pair of hangers is suspended by eight 1½-in. wire ropes, four to each hanger, which connect to a counterweight through equalizers. These suspending ropes extend from the hangers up through the truss posts, over sheaves above the panel points of the lift span, thence under grooved drums at the corners of the lift span, thence up and over sheaves on the tops of the towers, and so to the counterweights. The end floor-beam has only two ropes at each end. The pairs of drums at each end of the lift span are connected by transverse shafting, and the four drums are revolved simultaneously by motors and gearings placed in the machinery house at the center of the lift span. The lifting deck is operated through the friction of the operating drums on

the suspending ropes. Movement of the lifting deck does not affect the upper deck traffic.

The lift span which supports the lifting deck is suspended by sixty-four 2¼-in. wire ropes, sixteen at each corner, which pass up and over sheaves at the tops of the towers, and are connected through equalizers to the counterweights. The lift span and lifting deck are thus fully counterbalanced by two main counterweights for the lift span, and eight smaller counterweights for the lifting deck. In the lowest position the lift-span shoes bear on castings on the tops of sub-posts in front of the main tower posts. These sub-posts thus take live loads from both lift span and lifting deck, except the end panel loads of the latter, and carry also any unbalanced dead loads of the span or deck.

Operation of Lift Span.

The lift span is operated by four drums actuated through trains of gears by electric motors. Each drum controls two pairs of ropes, one pair of up-haul and one pair of down-haul, in arrangement substantially as described for the Interstate Bridge. The lifting deck and the lift span are each separately operated by two electric motors, but the two sets of machinery and the four motors are connected so that it is possible to operate either with any two motors. Controllers are of the magnet control series type. Limit switches for emergency are provided both for the lifting deck and the lift span. Most of the operating machinery is in a machinery house above the top chords of the lift span; the controller, indicators, and other equipment are in an operator's cabin below the machinery house, from which the upper roadway is visible. The locks close automatically, but are drawn by motors which are interlocked with the lifting machinery. Both the lift span and the lifting deck are controlled by hand-brakes.

For guidance during movement, both the lifting deck and the lift span have guides at each corner engaging on tracks on the towers. For the deck these are slotted jaws; for the span, double roller guides are backed by heavy springs which keep the longitudinal rollers constantly in contact with the tracks. The guide-tracks for the lifting deck do not extend above the lift-span shoes. When the lift span rises, the lifting deck is held tightly in contact with it so that when the span is reseated the deck guides are properly re-entered on their respective tracks. The guides for the counterweights are similar. Each group of deck counterweights has double jaw-guides engaging in turn on tracks on the towers and on short tracks on the main counterweights, so that, after the deck is fully lifted, its counterweights move with the main counterweights which have guide-tracks for the full height of movement. Tapered center castings near the lower limits bring both deck and span to exact lateral positions when seating.

At the floor joints between the lift and the fixed spans, steel floor-plates on the lift extend over and rest on the fixed spans when the span is down. Street-car and railway-track rails are extended across these gaps by manganese steel gap bars, which seat against the webs of the rails on the fixed spans, affording continuous tracks. There are heavy manually operated gates for roadways and sidewalks to shut off traffic during span movement. For each right-hand, street-car track there is a derailing switch connected to, and operated by, the adjacent gate.

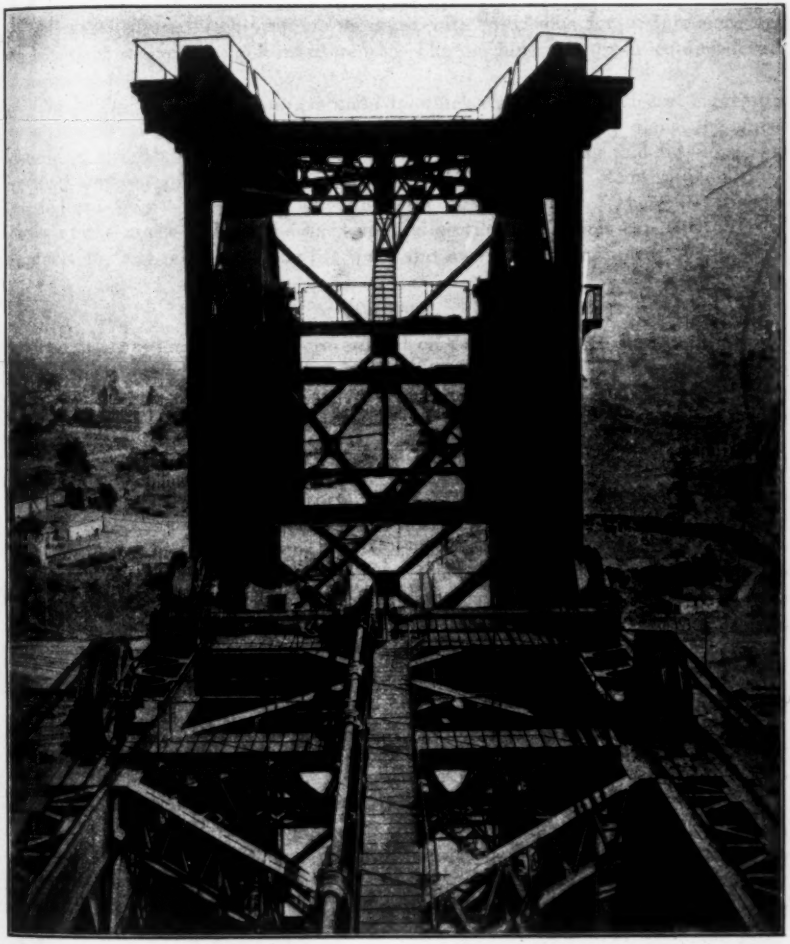


Fig. 12.—Harriman Bridge: View from Machinery House, Showing Portion of Top Deck of Lifting Span and Top of Tower.

With extreme movement, the main counterweights descend to within a few feet of the highway floor. The street-car trolleys, within the towers, are carried on movable longitudinal bars which are pushed down as the counterweights descend, and return to their original position as the counterweights ascend.

All parts of the machinery are designed with provisions for maintenance and replacement without serious interference. The machinery house is equipped with a traveling hand crane.

The bridge carries a 12-in. gas main in which continuous service at a pressure of about 60 lb. is maintained. From a section of pipe lying along the upper chords of the lift span two vertical sections of pipe are hung and telescope into vertical sections of larger pipe fastened to the towers and connecting to pipes carried out from shore on the lower decks of the fixed spans. There are stuffing-boxes at the upper ends of the fixed vertical sections. The gas can flow uninterruptedly for any position of the lift span, and during span movement.

Erection.

The two fixed river spans were erected on falsework, which was unusual only in that the piles were of single sticks from 90 to 130 ft. long. The floor system and laterals were placed by a locomotive crane but the trusses were erected by a floating derrick comprised of "sheer legs" 110 ft. high, mounted on the end of a large scow.

In order to avoid interruption of river traffic during construction, the lift span was erected on elevated falsework affording a vertical clearance of 116 ft. and a full 200-ft. horizontal clearance at water level. This falsework consisted of four wooden Howe trusses 120 ft. long which rested on eight wooden cantilever brackets built in front of, and attached to, the channel ends of the fixed spans. The brackets rested on the piers, were 82 ft. high, and overhung 52 ft. The front post of each bracket was of six 12 by 12-in. timbers bolted together. The brackets and trusses were framed on scows and hoisted into place, each unit complete.

A timber gallows frame, 150 ft. high, hinged at its base and restrained by guy lines, was placed successively near each end of this falsework and served to erect the towers and main sheaves. The correct locations of the tops of the towers were determined by a plumb line of piano wire. Adjustment was made by raising or lowering the back columns of the towers, after which the rivets of the joints connecting these columns to the top chords were driven.

The lift-span material was placed with an overhead gantry traveler of two bents, 96 ft. high, operated first on the falsework and, later, on special stringers on the cantilevers of the span itself.

The main counterweights were built in lowered position on forms supported on sand-boxes 18 in. high, resting on timber falsework extending up from the piers. The suspending ropes were placed over the sheaves and connected, and when the lift span was completed and the counterweights set sufficiently, the sand-boxes were opened, thus suspending the counterweights and relieving all the falsework under both span and counterweights. The machinery and operating ropes for the lifting span were placed and adjusted, and the span falsework was then removed.

The lifting-deck counterweights were built on low falsework supported on the upper decks of the fixed spans, and the ropes for suspending the lifting deck

were attached and laid in place over the sheaves and extending down into the various posts. The material of the lifting deck was brought on a barge to a position just below the lifting span, and each floor-beam was riveted to its two hangers. Each such frame was hoisted into place with the hangers inside the respective posts and connected to their suspending ropes. The stringers were then drawn up and erected between the floor-beams, and the other material was similarly placed.

When the steelwork of the lift span and lifting deck was finished, enough additional load of steel rails and other materials to compensate for the yet unlaid decks was hoisted to the span, and it was lowered to its seat. Guides, shoes, and locks of both span and deck were placed and adjusted with successive movements of the span. The flooring materials were placed on the span and deck in lowered position. The spans were lifted to clear the river traffic, and the decking was placed. As soon as the lower deck was completed and before the floor of the upper deck had been fully placed, railway traffic was turned on the bridge, as the necessity for relieving the old bridge was imperative.

Other Types of Lift Bridges.

To illustrate other contrasting types of vertical lift spans, in conclusion, Figs. 13 and 14 show a bridge built under great difficulties in Russia, and an inexpensive single-track girder span, 50 ft. long, near Harrisburg, Tex., respectively.

A single-track railway bridge for the Spokane, Portland, and Seattle Lines over the Willamette River, at Harrisburg, Ore., was originally built with three 200-ft. fixed spans and two 100-ft. half through girders (Fig. 15) at a cost of $170 000. It then marked the head of navigation on the river. Some years later the War Department directed that provision be made for river traffic. Machinery and lifting parts were added to the central span, towers to the adjacent spans, and the bridge was made operative, as shown in Fig. 16. This was done without interference with traffic. The cost (during the war period) of converting the fixed span into a movable bridge was $50 000. The tower extensions above the sheaves are for power and telegraph wires.

Personnel and Prices.

This bridge was designed and built under the direction of John D. Isaacs, M. Am. Soc. C. E., at that time Consulting Engineer of the Harriman Lines. Mr. George W. Boschke was then Chief Engineer, and George T. Forsyth, M. Am. Soc. C. E., was Bridge Engineer, of the Oregon-Washington Railroad and Navigation Company. The engineering work of design, preparation of complete detail plans and specifications, and of supervision of construction was carried out by the writer's then firm, and occupied the especial attention of John Lyle Harrington, M. Am. Soc. C. E. The foundations were constructed by the Union Bridge and Construction Company under the personal direction of L. S. Stewart, Assoc. Am. Soc. C. E., and H. K. Seltzer, M. Am. Soc. C. E. The metal superstructure was manufactured by the American Bridge Company. Albert F. Reichmann and F. W. Dencer, Members, Am. Soc. C. E., both being in close touch with the work. The erection of the superstructure, the construction of the deck and the completion of the bridge were in the hands of Mr. Robert Wakefield, of Portland, Ore.

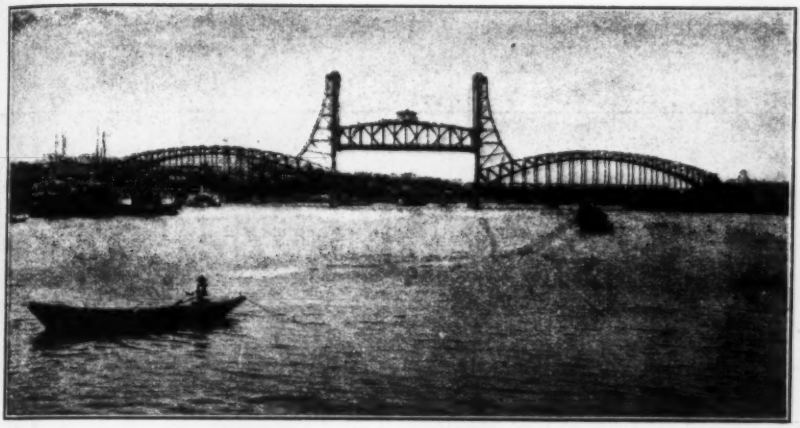


FIG. 13.—LIFT BRIDGE OVER DON RIVER AT ROSTOFF, RUSSIA, FOR VLADICAUCASE RAILWAY COMPANY. DOUBLE-TRACK LIFT SPAN 217 FT. LONG; FIXED SPANS 317 FT. LONG. BUILT BY UNSKILLED LABOR; COMPLETED IN 1917, AFTER RUSSIAN REVOLUTION.

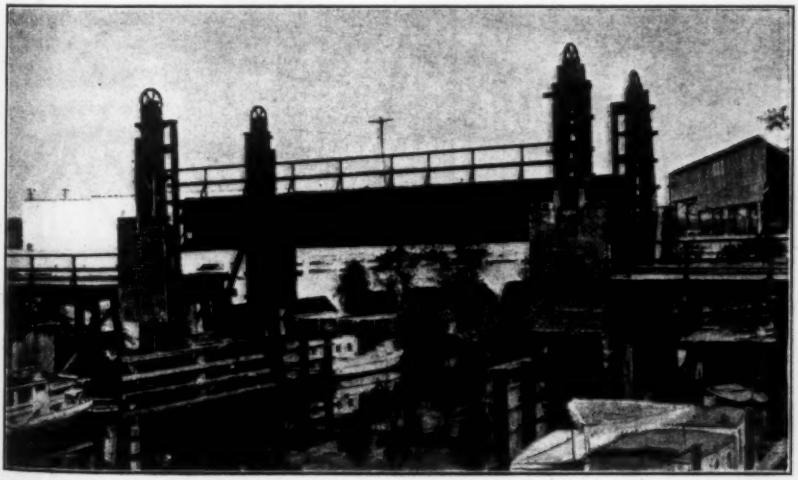


FIG. 14.—RAILWAY BRIDGE OVER BRAYS BAYOU, NEAR HARRISBURG, TEX. INEXPENSIVE LIFT BRIDGE 50 FT. LONG, WITH WOODEN TOWERS; LIFT SPAN IN RAISED POSITION 20 FT. ABOVE HIGH WATER.

FIG. 15.—BRIDGE OVER WILLAMETTE RIVER AT HARRISBURG, ORE., AS ORIGINALLY BUILT—THREE 200-FT. FIXED SPANS.

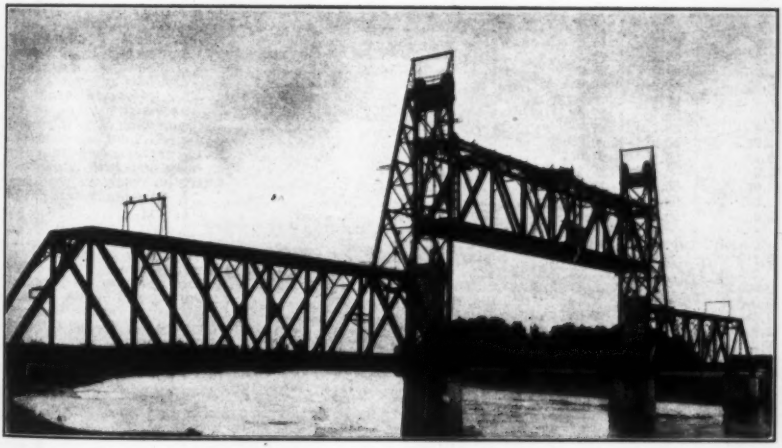


FIG. 16.—CENTRAL SPAN OF ABOVE BRIDGE MADE OPERATIVE.

Tables 3 and 4 give unit prices of the principal items. These units are the cost to the Company, and should represent contractor's cost plus profit. The cost of unclassified work in the substructure was $41 912.37, and in the superstructure $5 694.66. The large item of unclassified work in the substructure is the cost of leveling the bed-rock foundations for the main river piers which could not be pre-determined. Right-of-way costs and certain other charges are omitted.

TABLE 3.—Unit Costs of Substructure, Harriman Bridge.

Mass of base, Piers Nos. 1 and 2, including timber, concrete, excavation, sinking, etc., per cubic yard	$16.00
Mass of base, Piers Nos. 3 and 4 (river piers), per cubic yard	11.25
Concrete in shafts, Piers Nos. 1 and 2, including forms, coffer-dams, pumping, etc., per cubic yard	7.50
Concrete in shafts, Piers Nos. 3 and 4, per cubic yard	8.00
Concrete in base, Abutment *B*, including excavation, etc., per cubic yard	17.00
Concrete in shaft, Abutment *B*, per cubic yard	7.50
Concrete piles, per linear foot	1.32
Timber piles for amounts below bases, per linear foot	0.48
Timber piles for cut-offs, per linear foot	0.154
Cutting edges, per pound	0.09
Timber in cribs, per 1 000 ft. B. M.	48.00
Cement, per barrel	1.90

TABLE 4.—Unit Costs of Superstructure, Harriman Bridge.

	Material at shop.	Erection.	Total.
	Cents per Pound.		
Truss spans	2.61	1.00	3.61
Towers	2.61	1.40	4.01
Columns and bents of highway viaduct	2.89	0.60	3.49
West railway approach	2.48	0.60	3.08
Girders, highway viaduct	2.68	0.60	3.28
Counterweights, metal	2.82	1.40	4.22
Sheaves, 14 ft. in diameter	6.72	1.50	8.22
Sheaves, 76 ft. in diameter	7.78	1.50	9.28
Equalizers and pins	9.90	2.50	12.40
Wire ropes and sockets	13.10	2.50	15.60
Sheaves and drums on span	7.78	1.50	9.28
Operating machinery on span	8.84	1.50	10.34
Street railway material	2.49	1.60	3.09
Gas pipe	8.93	1.00	9.93
	Miscellaneous Costs.		
Concrete in counterweights, per cubic yard	.....	.....	$11.00
Motors and electrical equipment (lump sum)	.....	.....	19 409.00
Machinery house (lump sum)	.....	.....	2 000.00
Lighting system (lump sum)	.....	.....	2 500.00
Creosoted timber, per 1 000 ft. B. M.	$30.00	$15.00	45.00
Creosoted paving, per square yard	1.00	1.50	2.50
Water-proofing per square yard	1.00	1.86	2.86
Concrete base and curbs, per cubic yard	2.19	5.00	7.19
Cements for counterweights, per barrel	.....	.....	1.90
Freight, per 100 lb	.....	.....	0.80

The bridge charter and contracts provided that the City should pay to the Company for the use of the highway portion of the bridge a rental based on the additional money spent to provide a combination bridge above what would have been required for an exclusive railway bridge. Careful estimates of cost were made for a bridge exclusively for railway traffic and for one exclusively for highway traffic, each to be at the same location and to afford separately the same facilities now given by the combined bridge. Omitting minor items affecting each equally, these estimates are as follows:

Actual cost of combined bridge	$1 700 000
Estimated cost of bridge for railway traffic only	880 000
Estimated cost of bridge for highway traffic only	1 090 000
Additional cost of building a combined bridge	820 000
Saving in first cost by constructing one combined bridge instead of two independent bridges, about	270 000

The total amount saved by the combination would include saving in operation, maintenance, money value of less obstruction to river traffic, etc.

Excellent records of movement of all the Willamette River movable bridges are available, and make it evident that the double movement feature of the lift span saves highway delays averaging more than 3 hours per day, or nearly 5 months of 8-hour days per year of actual bridge service, over the service which could have been afforded by a double-deck swing bridge.

The service of the structure has been excellent throughout and its operation reliable. The mild winters in Portland permit river navigation throughout the year, so that the lift span and lifting deck are in constant service.

DISCUSSION

D. B. Steinman,* M. Am. Soc. C. E.—Mr. Howard has rendered a service to the Bridge Engineering Profession by his valuable paper on vertical lift bridges. The information presented therein should be of assistance to engineers engaged in making bridge layouts in which movable spans must be provided.

One phase of the subject, however, should receive more attention than has been given to it in the paper, that is, the question of the economic limits of the vertical lift in competition with other types of movable bridges. More definite information on this subject would be appreciated by those in charge of bridge projects, as it would guide them in the selection of the type of movable structure for the preliminary layout, and would save valuable time otherwise wasted in considering types which may be decidedly uneconomical.

The author writes on this subject, as follows:

"For narrow openings where vessels with lofty masts must be accommodated a bascule is preferable; but where the vertical clearance required does not exceed the channel width, the lift span is found to be nearly always cheaper than the swing bridge or bascule, notably for long spans."

The speaker has made an independent study of this question, and has found a more definite relation between span and height for the economic limit of vertical lift bridges. This relation is given by the simple formula:

$$H = \tfrac{1}{2} L \qquad (1)$$

where H = the height to be lifted, in feet; and
L = the length of the lift span, in feet.

A straight line may be plotted as a graph of this equation. For all combinations of span and lift represented by points falling below this graph, the vertical lift will be more economical than any other type of movable bridge. For points falling above the graph, the economy will generally be in favor of the bascule type.

It is recognized that this comparison of movable bridges will be affected by various conditions in different projects, such as depth and character of foundations, square or skew layout, arrangement of flanking spans, etc. Where the conditions are specially favorable to the vertical lift, the economic limit for that type may be raised to:

$$H = \tfrac{1}{2} L + 30 \qquad (2)$$

The graph (Fig. 17) of Equation (2) is a straight line above and parallel to the first graph. The parallel strip between the two graphs contains all points of equal cost for the bascule and vertical lift types. For layouts with shallow foundations and comparatively inexpensive substructures, the lower boundary, Equation (1), should be used. For skew layouts, layouts having comparatively expensive substructures, and layouts having some specially favorable arrangement of flanking spans, the upper boundary, Equation (2), should be utilized. For points falling in the strip between the two graphs, a special study or comparative estimate is required to decide which type of movable span should be adopted.

* New York City.

The height, H, in Equations (1) and (2) is the actual height of movement to be provided. It averages about 80% of the total vertical clearance to be provided, the other 20% being the under clearance when the span is in its fixed position.

The maximum value of H which may be required is about 120 ft. (corresponding to a vertical clearance of 150 ft.). The corresponding values of the span length, L, are 240 and 180, by Equation (1) and Equation (2), respectively. Accordingly, the economic limit of the bascule is between 180 and 240 ft., depending on the foundation conditions; and there would appear to be no justification for considering the bascule for any span exceeding 240 ft.

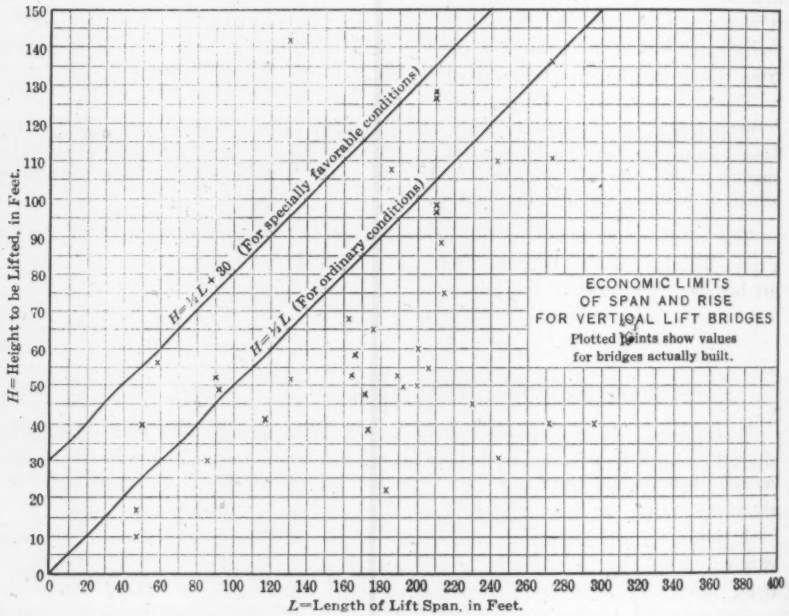


FIG. 17.

These conclusions may be summarized, as follows:

1.—Where the span length is greater than 240 ft., the vertical lift bridge is the most economic type. If the foundations are expensive, this limit may be reduced to 180 ft.

2.—For spans of less than 240 ft., if the height of the lift does not exceed $\frac{1}{2} L$, the vertical lift bridge is the most economic type. If the foundations are expensive, or the bridge is on a skew, this limit may be raised to $\frac{1}{2} L + 30$ ft.

Other conditions, beside total first cost, may determine the selection of the vertical lift type. Such conditions are:

1.—Where it is required to provide for possible future shifting of the channel by making the fixed and movable spans interchangeable.

2.—Where the bulk of the river traffic consists of smaller vessels, so that the bridge, if of the vertical lift type, will not have to be fully opened except on rare occasions.

3.—Where a double-deck structure is required.

4.—Where it is desired to carry overhead water mains, electric conduits, or steam pipes across the opening.

These considerations may be deciding advantages in favor of the vertical lift type, even in cases of doubtful economy.

The foregoing comparison has been limited to the bascule and vertical lift types, as no other type of movable bridge need be considered in economic competition. For the swing bridge, it is generally agreed, there is no longer any economic justification since the bascule and vertical lift types have been developed.

For the comparison of the vertical lift with other types of movable bridges, the speaker has prepared the following list of advantages, some of which have been mentioned by the author.

1.—Within the limits previously defined, the vertical lift bridge is more economical than the other types, both in first cost and in cost of operation. To this saving, the following advantages over the bascule type are contributing elements:

(*a*) The simple, usual construction insures lower unit prices. The heavy and expensive moving parts of bascule bridges are eliminated.

(*b*) Both counterweights together weigh the same as the lift span, instead of three or four times as much; hence, they cost less, the foundations required to support them are less costly, and they require less power to move, than those of the bascule type. These become very important items when the span is long and when the foundations are deep or otherwise expensive.

2.—The vertical lift bridge possesses the further advantage that it is a simple span in which all stresses are determinate, and the same as those of a fixed span.

3.—The vertical lift is the only type economically adapted to a skew crossing. Under the same conditions, the bascule type requires either an expensive special construction, or else the added cost of an otherwise unnecessary lengthening of the movable span, in order to secure the same channel clearance.

4.—The vertical lift is the only type economically adapted to a future change of elevation of grade.

5.—The vertical lift type is as rigid under traffic as a fixed span. This is not the case with the rolling bascule, nor with the trunnion bascule in the longer span lengths which require double-leaf construction.

6.—The vertical lift type is economically adapted to construction on a grade.

7.—A vertical lift bridge can be paved like a fixed span, since the floor is always horizontal. Track construction and maintenance are also simpler than on a bascule bridge.

8.—With the vertical lift type, spans of any length may be built, the longer the span, the greater the economy over other types.

9.—The vertical lift type exposes small areas to the wind. In the bascule type, the wind acting against the exposed area of the floor when the span is open

imposes a heavy load on the machinery and may cause serious delay or interruption in operation.

10.—With the vertical lift type, only one operator is required, no matter how large the open channel may be.

11.—It is practicable, by simply adding towers, counterweights, and machinery, to convert an existing fixed span into a vertical lift bridge.

12.—Where the necessity of making the span movable can be deferred, the vertical lift type affords a large saving in first cost, as the immediate cost of providing for the future transformation is reduced to a minimum.

13.—In alluvial rivers where the channel is likely to shift, the towers and counterweights may be moved at small cost, and any other span in the bridge made a lift span, thus avoiding the expense of controlling the channel.

14.—The vertical lift type is economically adapted to double-deck construction. Furthermore, by providing for independent lifting of the lower deck, large economies in both construction and operation are secured.

15.—The vertical lift bridge does not have to be raised to full height except for the rare passages of high-masted vessels. This fact permits an economy in cost and time of opening.

16.—With the same length of truss span, the vertical lift type affords a greater channel clearance than the bascule type, on account of the unavoidable overhang and the large piers of the latter type.

17.—The vertical lift is the only type of movable bridge producing fixed and unvarying dead load reactions on all the piers. This is an important desideratum in the case of foundations subject to settlement under the impact of heavy varying load. Furthermore, provision for the moving and variable reactions in the different bascule types involves additional expense for the piers and foundations.

18.—The troubles in design, fabrication, and operation arising from the track and segment in rolling bascules, and from the heavy trunnions in trunnion bascules, are eliminated in the vertical lift type of movable bridge.

19.—The rolling bascule and some types of trunnion bascules necessitate an open space in the floor when the bridge is open. This is dangerous to bridge traffic and causes trouble on account of the collection of dirt and water.

As the advantages of the vertical lift bridge become more generally recognized, the use of this type will undoubtedly become widely extended.

SHORTRIDGE HARDESTY,* ASSOC. M. AM. SOC. C. E.—The speaker has been especially interested in this subject, because he has had the opportunity to be in close touch with the development of the vertical lift bridge: from 1908 to 1915, with the firm of Waddell and Harrington, and, from 1915 to date, as Designing Engineer for J. A. L. Waddell, M. Am. Soc. C. E.

To those who are interested in this subject the speaker would recommend a paper entitled "The Mechanical Features of the Vertical Lift Bridge", by H. P. Van Cleve, M. Am. Soc. C. E., presented at the Annual Meeting of the American Society of Mechanical Engineers in December, 1918. This article covers a different phase of the subject from Mr. Howard's paper, in that it does not discuss the general features to any extent, but goes into the mechanical equip-

* New York City.

ment in detail, elaborating some of the points touched on by the author. It also treats fully of the unusual problems encountered in the erection of the bridge over the South Branch of the Chicago River for the Pennsylvania System (the twelfth in Table 1). The detailing of large built sheaves (mentioned by Mr. Howard)* is thoroughly treated in Mr. Van Cleve's paper and the accompanying discussions.†

A general descriptive paper on the subject was presented by Dr. Waddell in 1915 to the Pan-American Scientific Congress. It contains illustrations of nearly all the vertical lift bridges built prior to that date.

The Committee on Iron and Steel Structures of the American Railway Engineering Association has a Sub-Committee on the Relative Economics of Various Types of Movable Spans, of which B. R. Leffler, M. Am. Soc. C. E., is Chairman. This Sub-Committee has investigated the subject of movable bridges quite extensively, and is now preparing specifications for their design, preliminary drafts of which have already been published. As a result of studies made in connection with his work on this Committee, Dr. Waddell has just finished a paper on the "Economics of Movable Spans", which will be published shortly by the American Railway Engineering Association. This paper will treat quite extensively of the vertical lift bridge, especially in reference to its costs as compared with those of other types of movable spans. The subject has been analyzed as thoroughly as the data at hand would permit.

The author makes the statement that "where the vertical clearance required does not exceed the channel width, the lift span is found to be nearly always cheaper than the swing bridge or bascule, notably for long spans." The findings of Dr. Waddell's investigation are in exact agreement with this statement. It may not always hold for a very light span; but for long, heavy spans, the vertical lift will usually be found to be cheaper, even when the vertical clearance is somewhat greater than the channel width.

The conclusion can also be drawn that long span lengths favor the vertical lift type. This is due to the fact that the costs of the special features of the vertical lift span are practically independent of the span length, the only difference being that due to a slight effect of wind loads on the towers of light spans. Thus, for a 1 000 000-lb., lift bridge, the cost of the towers, machinery, counterweights, etc., will be almost exactly the same for a 200-ft. span as for a span of 100 ft. In the case of a bascule weighing 1 000 000 lb., the cost of the trunnions, machinery, counterweights, towers, counterweight trusses, etc., will be nearly twice as great for a 200-ft. span as for a span of 100 ft.

It can also be stated that heavy construction favors the vertical lift. This is due to the fact that the bracing and the rear legs of the towers weigh nearly as much for a light span as for a much heavier one. The cost of these tower bracing members is proportionately small for a heavy span but relatively much greater for a light one.

The presence of flanking truss spans on each side of the movable span favors the vertical lift. The rear legs of the towers rest on the flanking spans without producing any material stresses therein; whereas, in the case of the bascule, it is necessary to add an extra pier, to carry the heavy weight of the moving leaf

* See p. 583.

† *Transactions*, Am. Soc. Mech. Engrs., Vol. 40 (1918), p. 1017.

and counterweight on the flanking span, or to move the channel pier back until the trunnion comes over the pier. Any of these solutions involves considerable expense.

There are several factors which operate to keep down the unit costs of the vertical lift bridge and make it very desirable from the shop standpoint. In the first place, there is little special structural work, the lift span itself differing only slightly from an ordinary span, and the towers presenting no unusual fabrication problems. In the second place, the machinery layout is simple and contains no pieces involving difficult construction. In the third place, the machinery and structural portions are entirely separate, thus obviating the expensive fitting of machinery parts into large structural members. Furthermore, the machinery can be fabricated in one shop and the structural metal in another, if necessary. Holes for connecting the machinery to the structural work can be either drilled to templates, or else drilled or reamed in the field after assembling.

The author refers, near the bottom of page 586, to the unbalanced weight of the suspending ropes. If not too great, this lack of balance is an advantage, for it provides a positive reaction tending to hold the span down to its seat.

That the vertical lift type is thoroughly satisfactory is attested by the fact that it has gradually won its way into favor against prejudice. Many bridge engineers who were opposed to its use several years ago have come to favor it. The bridge engineer of one of the large railway systems, who had structures of this type under his care, once stated to the speaker that, to his mind, it should be the ideal type of movable span in that whether seated or raised it acts exactly like any fixed span, being subject to no stress variation of any kind. The engineer of another large system wrote to the speaker not long ago that, in his opinion, the vertical lift was as good as any other type, and better than many of them, and he also stated that the vertical lift spans were the most satisfactory movable spans on that system from the standpoint of both operation and maintenance.

The list of lift bridges given by the author in Table 1 should be supplemented by those in Table 5, which have been designed by Dr. Waddell since 1914.

The vertical lift has been handicapped in the East on account of the fact that there have been no examples east of Chicago, Ill., and Louisville, Ky., with the exception of some small low-lift structures over canals. As a result, it has been difficult to persuade prospective clients to adopt that type. With the completion of the 300-ft. span at Portsmouth, N. H., and the structure at Trenton, N. J., this handicap will be removed.

The Portsmouth Bridge, now being constructed over the Piscataqua River by the States of Maine and New Hampshire and the Navy Department, presents a number of interesting construction problems, which are being solved by the engineers of the American Bridge Company, the contractor for the superstructure, and the speaker. The river is about 900 ft. wide, and about 80 ft. deep for more than half its width. It is only a few miles from the ocean; and as there is a large bay up stream from the site, heavy tidal currents exist. The bottom is rock overlaid by only a few feet of soft material, so that falsework is out of the question except near each shore. The layout consists of three 300-ft. spans, the central one being the lift span. Two erection schemes have been considered: First, building all three spans on falsework near one bank about ½ mile from

the site, and floating them into position; and, second, erecting the end spans on falsework and cantilevering the lift span out from both sides. Owing to the dangers attending the use of the first scheme at this location, the second plan is preferred. The spans are of riveted construction, but the central bottom chord member in each truss of the lift span is pin-connected. This member will be omitted until all the other joints are riveted, and the towers, ropes, machinery, and counterweights are in position, in order to provide a restricted channel for the infrequent river traffic. When all is ready, the omitted members will be put in, the abutment shoes of the fixed spans jacked up to relieve the stresses in the temporary cantilever members over the piers, and these members removed. The span can then be lifted at once.

TABLE 5.—DATA ON ADDITIONAL LIFT BRIDGES.

Bridge over	Kind of traffic.	Length of lift span, in feet.	Maximum clear height above high water, in feet.	Normal height of movement, in feet.	Total moving load span, counterweights, etc., in tons.
Ohio River Bridge, at Louisville, Ky., for the Louisville Bridge Company (Pennsylvania System)	Double-track railway	264	75	39	3 200
Cross Bayou Bridge, at Shreveport, La., for the Caddo Levee Board	Highway	54	66	30	280
Tallahatchie River Bridge, near Charleston, Miss., for the Lamb-Fish Lumber Company	Single-track railway	76	63	46	160
Osage River Bridge, near Jefferson City, Mo., for Cole and Osage Counties (State and Federal Aid)	Highway	100	47	33	175
Piscataqua River Bridge, at Portsmouth, N. H., for the Piscataqua River Bridge Commission	Highway and street-car	298	150	128	1 800
Southard Street Bridge, Trenton, N. J., for County of Mercer	Highway	70	50	28	680
Warrior River Bridge, Tuscaloosa, Ala., for Alabama State Highway Department	Highway and street-car	186	50	36	860
Cowlitz River Bridge, Kelso, Wash., for Washington Department of Public Works.	Highway	107	55	47	660

The counterweights were designed in the usual manner, that is, with steel frames surrounded by concrete. In working out the erection plan, however, it was found advisable to be able to swing the span and raise it without having to wait for the counterweight concrete to set, and also to be able to add to the counterweights later when the deck was being placed, without preventing the operation of the span. For these reasons it has been decided by the contractor to cast each counterweight in a steel box suspended from the equalizers, making the box strong enough to carry the wet concrete. The saving in erection cost and time will be sufficient to pay for the extra metal in the steel boxes.

One advantage of the vertical lift bridge, which has been referred to by the author, is the fact that it will accommodate a future change in grade. On the Ohio River Bridge, at Louisville, Ky., the tracks are now on a grade; but it is expected that at some future time the grade will be raised considerably, and made level. The lift span was built with the chords level; but the floor system was set to the present grade, the bottoms of the portals and sways being placed

high enough to accommodate the future grade. When the change in grade is made, the floor systems in the towers will be brought up to the new grade, the floor system on the lift span will be made horizontal, and the span will then be lifted vertically to the new grade, the lift-span shoes being raised as required.

It is noteworthy that the original counterweight ropes of the Halsted Street lift span, Chicago, Ill., which were put in place about twenty-eight years ago, are still in use. The objection is frequently heard that the ropes will wear out rapidly, so that there will be a large expense for renewals. The experience with the Halsted Street Bridge shows that this is not correct, so that there need be no fear that the counterweight ropes will require renewal except at very long intervals. The question is somewhat different with the operating ropes. They carry none of the weight of the structure, and their renewal is an easy matter. Sheaves and drums sufficiently large could be used so that these ropes would not need renewal any oftener than the counterweight ropes; but it will be found to be economical to use smaller drums and sheaves, and plan to renew the operating ropes occasionally.

THOMAS E. BROWN,* M. AM. SOC. C. E.—The speaker is reminded that the earliest lift bridge with which he was connected was built in Chicago, Ill., nearly thirty years ago. Its designer, J. A. L. Waddell, M. Am. Soc. C. E., engaged the speaker to prepare a design for structural sheaves suitable for the work, and this design was quite similar to the sheaves shown by the author. Later, the construction and installation of the machinery were entrusted to a company of which the speaker was Engineer, and it is his impression that the structural sheaves were not used, as sheaves built up of cast iron and steel were found to be somewhat cheaper in first cost, the diameter being only about 12 ft.

At that time, though slightly less than thirty years ago, electric motors were in their infancy, and it was thought to be too experimental to use them in connection with a structure which in itself was considered to be somewhat of an experiment, and steam hoisting engines were installed. These engines were in use for many years and, when worn out, were replaced by electric motors. This is interesting as showing how very modern is the use of electric motive power for the operation of movable bridges; even in Buffalo, N. Y., the home of electricity, several bridges were built, one as late as 1905, which were operated by steam.

Even after Dr. Waddell's design had been accepted, many engineers and the general public had so little faith in his novel proposition that the authorities, in gambling parlance, had "cold feet", and endeavored to avoid making the supposed experiment, and it was only when they were compelled to do so in view of the contract which could not well be cancelled, that the work was allowed to proceed.

The author's statement of the economic limits of the lift bridge is thought to be more nearly correct than Dr. Steinman's formula, though the difference is not very great.

Although the owner of several bascule bridge patents and, therefore, naturally prejudiced in their favor, the speaker some years ago made comparisons of the various types of bridges and came to the general conclusion that when the clear height required above high water did not exceed the span length, the lift bridge was less costly than the bascule type. He believes that there has been no later

* New York City.

development of the bascule bridge which would modify this general conclusion, and, therefore, is convinced that the author's statement relative thereto is substantially correct. Mr. Howard's paper is the most interesting one on the subject yet presented to the Society.

G. LINDENTHAL,* M. AM. SOC. C. E. (by letter).—This able paper is a welcome addition to the literature on movable bridges, on which information is still greatly scattered.

There is one point on which the author, from experience, may be able to give reliable data; that is, the time and cost of operation. The only reference in the paper to time of operation is an indirect one, namely, when the author says, (on page 601), "the maximum movement is 50 ft., which is traversed in less than 1 min." The meaning in this sentence is not clear. How much time is required for lifting or closing and how much time for the other operations connected therewith?

The cost of operation may be divided into overhead cost and the cost depending on the number of openings. A comparison of cost in the various types of movable bridges would be useful at hearings before authorities, when the question of movable or non-movable high bridges is under consideration.

ARTHUR G. HAYDEN,† M. AM. SOC. C. E. (by letter).—In this discussion the writer will attempt to supplement Mr. Howard's paper on the subject of lift bridges by describing a method of operation, adapted to small structures, which was used recently for a number of bridges with spans of about 150 ft. and vertical lifts of about 15 ft., on the New York State Barge Canal.

These bridges are of the concealed counterweight type and, being of small lift, are operated at one end only. The end verticals of the bridges are built as lifting posts and extend below the bottom chords into pits in the abutments. In these pits the counterweights, counterweight sheaves, and all the operating machinery are placed. The bridge superstructure carries no load from operating machinery. In closed position the bridges have the appearance of being stationary, and there are no unsightly towers, overhead counterweights, or machinery parts in view.

The bridges are driven by electric power applied, through a train of gears, to lifting-post racks at one end only. They are maintained in a horizontal position, under the action of this eccentrically applied force, by equalizing cables, the function of which will be explained. This method of operation is the feature which will be described in detail by the writer.

Fig. 18 shows how an uncounterweighted part of the weight of the bridge is held in equilibrium by means of a tooth reaction at the rack and the action of the equalizing cable, A. An additional unbalanced upward force, P, applied at the rack will, of itself, tend to impart to the bridge a motion of translation vertically upward, combined with a motion of left-hand rotation about its center of gravity. However, the equalizing cables, A, prevent rotation in this direction, and the structure is constrained to move vertically upward without rotation. An opposite arrangement of cables, B, shown in Fig. 19, prevents right-hand rotation and constrains the bridge, under the action of an unbalanced downward force applied at the rack, to move vertically downward without rotation.

* New York City.
† Bronxville, N. Y.

Fig. 19 shows, in diagram form, the principle of operation of these bridges. In actual construction, the deflecting sheaves, *A* and *B*, at either end of the bridges, are carried on the same pin so that the cables, *A* and *B*, run within the bottom chord in the same horizontal plane with the chord center line. The counterweight sheaves (shown above the truss in the diagram for clearness of illustration) are placed in the counterweight pits below the plane of the bridge floor in its lowered position.

The Barge Canal bridges are slightly over-counterweighted so that in an emergency, such as failure of power, they may be released from the operating machinery and allowed to rise of their own accord and thus prevent damage to the superstructure from approaching boats. Elastic buffers are provided in the bottoms of the pits, beneath the counterweights, to absorb impact in such emergency.

Such parts of the equalizing cables as pass over deflecting sheaves during any part of bridge travel are of wire rope, but the horizontal portions along the bridge which do not pass over deflecting sheaves during any part of bridge travel, are of steel pipe running on idlers.

The counterweights are of concrete extending the width of the bridge at either end and are beneath the floor of the pit abutments in all positions. Part of the counterweight construction consists of removable blocks, 1 cu. ft. each in size, which allow for counterweight adjustment.

The counterweight cables are connected to the short cantilever ends of horizontal lifting beams attached to the lower ends of the lifting posts.

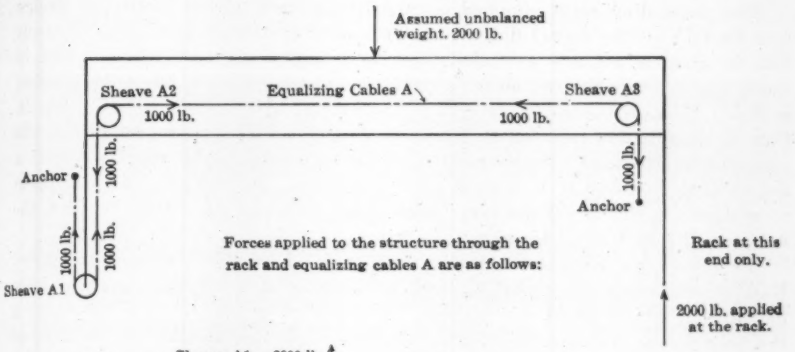
Pedestrian travel can be maintained while the bridge is raised by a stairway at each end of the bridge, leading to the sidewalk in its raised position. Traffic is safeguarded by fences attached to the lifting posts beneath the bridge floor, which close access to the Barge Canal when the bridge is raised.

HARRY G. HUNTER,* ASSOC. M. AM. SOC. C. E. (by letter).—Completed structures of the magnitude and importance of those described in this paper are, from their permanence, monumental in character. They represent during later years the achievements of the days of their inception. They stand as finished products—the fruition of the combined efforts and endeavors of many men and forces.

Not infrequently the engineer or builder of a structure finds his greatest interest and fascination, not in contemplating the completed result of his labors, but in meeting and overcoming those difficulties which arise during the prosecution of the work and incidental to it. The writer takes this opportunity to recount two instances of this nature from his experience, which he believes may be of interest, if not of value, to those who may encounter similar problems. One of these relates to the replacement of the suspending cables of a lift bridge, the other to the erection with primitive equipment of a heavy structure, reputed to be the longest movable span in Europe.

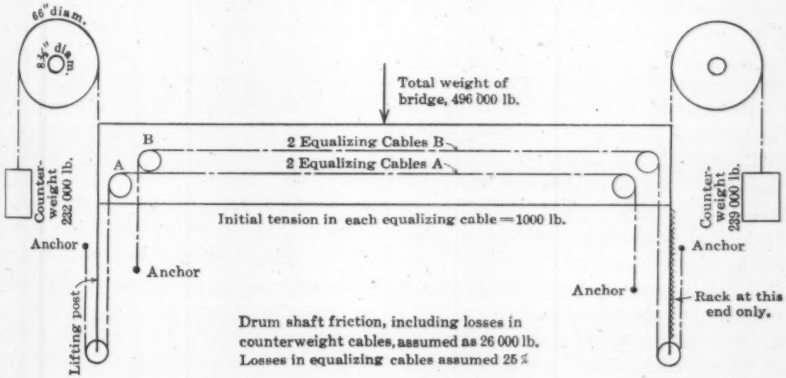
Replacement of Suspending Cables.—In his discussion of the qualities of wire ropes, the author states that with suitable connections the replacement of suspending ropes can be accomplished without interference with traffic or bridge movements. To the best of the writer's knowledge such renewal has been necessary on only one structure. A brief description of the method used in that case follows.

* Kansas City, Mo.



Sheave A1,	2000 lb. ↑		
Sheave A2,	1000 " ↓	Sheave A2,	1000 lb. →
Sheave A3,	1000 " ↓	Sheave A3,	1000 lb. ←
Rack,	2000 " ↑		
Resultant,	2000 lb. ↑ balances 2000 lb. weight		

FIG. 18.



Bridge Going Up

Far End				Machinery End			
Counterweight	232 000	Bridge	231 000	Counterweight	239 000	Bridge	238 000
Cables A	14 000	Friction	13 000	Cables B	1 500	Friction	13 000
		Cables B	2 000	Rack	29 500	Cables A	19 000
	246 000		246 000		270 000		270 000

Bridge Going Down

Counterweight	232 000	Bridge	231 000	Counterweight	239 000	Bridge	238 000
Friction	13 000	Cables B	16 000	Friction	13 000	Cables A	1 500
Cables A	2 000			Cables B	21 000	Rack	33 500
	247 000		247 000		273 000		273 000

FIG. 19.

The suspending ropes supplied for the lifting deck of the Harriman Bridge (see Part IV of the paper) did not conform in every respect to the specifications but, for reasons not now pertinent, they were tentatively accepted and used in the work. At the end of about a year the unsuitability of the ropes became manifest through the appearance of a few broken wires in the outside strands. Careful examination revealed no imminent danger, hence no immediate effort was made at replacement. Subsequently, a fire damaged eight ropes to such a degree that they had to be changed for new ones. This opportunity was taken to replace completely all the lower deck ropes with new ones fully meeting the requirements of the specifications.

As the author recites, the traffic accommodated by this bridge is enormous. Railway traffic across the lifting deck was such that the longest period of the day during which the deck could be taken out of service was 55 min. during the noon hour. The scheme of replacement was devised so that the work could be performed during several successive days, without the use of falsework or other temporary construction that would interfere with the use or operation of the structure. Traffic on the upper deck was not disturbed at any time.

The counterweights for the lifting deck are of the "sectional" type, each counterweight being composed of four sections, each section to balance the dead load concentrated at the corresponding panel points of the lifting deck. The three sections balancing the intermediate panel loads have four ropes attached to each end, that balancing the end panel has two ropes at each end. The scheme for replacement involved the changing of ropes on each section in turn, although only a part of the ropes on any section could be changed during one day.

Preparatory to changing a set of ropes, rope slings of ample strength to support the heaviest section of counterweight were connected to the tops of the towers and were of such lengths that their lower ends stood at about the elevation of the top of the counterweight when the lifting deck was in its lowest position. Attached by pins to the slings were two forged steel bars about 4 in. square by 5 ft. long which had, in their lower ends, pin holes bored to fit the projecting ends of the pins connecting the main equalizers to the counterweight sections. While these slings were being erected aloft, the new suspending ropes were being laid out on the highway deck ready to be hauled into place, and the load line from a hoisting engine placed on the highway deck was passed to the top of the counterweight. (See Figs. 20 and 21.)

All these preparatory measures being completed, everything was held in readiness to begin work as soon as the bridge was available. As soon as the lower deck was released from railroad service during the idle period, the lifting deck was raised somewhat, and the bars were attached to the equalizer pins. Then the deck was further lifted until the full weight of the section rested on the slings and the suspending cables became entirely slack. This movement was accomplished by the operating machinery of the lifting deck working against the unbalanced load of one section of counterweight. When the ropes were slack, a given rope was disconnected at both ends, the socket at the counterweight end was connected to the line from the hoisting engine, and the socket at the hanger end was attached to the socket on the counterweight end of the new rope. The hoisting engine then pulled the old rope down and, in doing so,

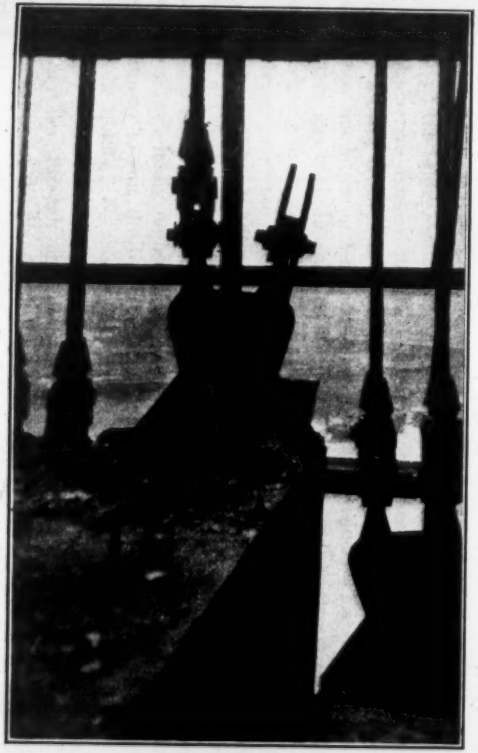


FIG. 20.—REPLACING SUSPENDING CABLES, HARRIMAN BRIDGE, PORTLAND, ORE.

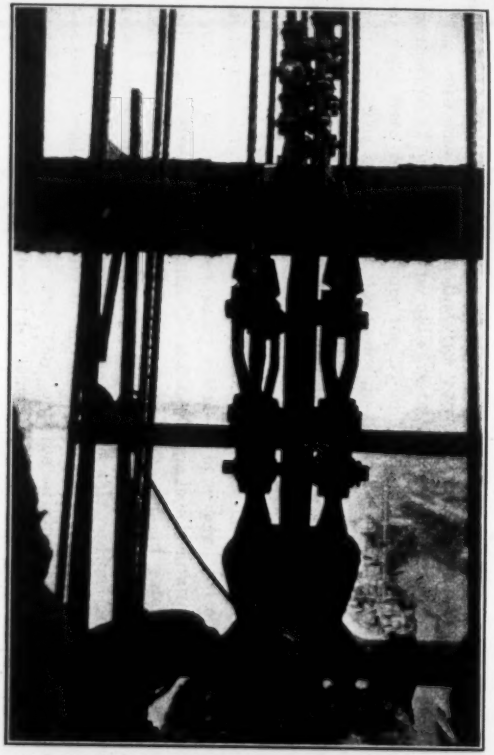


FIG. 21.—REPLACING SUSPENDING CABLES, HARRIMAN BRIDGE, PORTLAND, ORE.

threaded the new rope in its place. The pin connecting the new rope to the hanger was entered and connection also made at the counterweight end, after which the deck could be lowered, the rope slings released, and the bridge was again ready for trains.

The work was handled so that two corresponding ropes, one at each truss, were changed at one time. The ordinary task for one period was two ropes, but four ropes were replaced during the 55-min. period under especially favorable conditions. The entire work proceeded in an orderly and systematic manner, and the results were eminently satisfactory, not only in the accomplishment of the task, but also in the complete absence of delays or inconvenience to traffic.

Due to permanent and inelastic stretch of the ropes caused by the tightening of the strands under service conditions, the new ropes, although of the same manufactured lengths as the older ropes, were somewhat shorter than the latter. To overcome this difficulty, temporary extension bars were used on all new ropes attached to a given section until the last new rope had been attached to it, after which the deck was further lifted and all eight sets of extension bars were removed simultaneously and the ropes attached directly to the equalizers.

Subsequently, tests were made on the questionable portions of the old ropes. These tests revealed no failure of any test piece to meet the ultimate strength required by the specification and bore out the original conclusion that the removal could have been postponed for some time without undue risk.

The Don River Bridge.—The Don River Bridge No. 1, illustrated in Fig. 13, was begun in 1915 and completed in August, 1917. It formed one part of a very extensive undertaking embracing terminal and freight-handling improvements undertaken by the Vladicaucase Railway Company to relieve congested conditions at Rostoff-on-Don. The war greatly delayed the execution of the programme, and finally the revolution of 1917 caused the suspension of all work with the exception of the bridge, which was completed largely as a military necessity.

The structure crosses the Don River at the head of the Rostoff yards. It accommodates double-track railway and pedestrian traffic, the former between the trusses, the latter on 5-ft. cantilever brackets outside. This rather unusual combination was made in order to keep the seemingly endless stream of peasants, usually bearing heavy burdens, from using the track space.

The bridge is of the through type and consists of a central lift span similar to that of the Columbia River Interstate Bridge (Part II of the paper) and two tied arch approach spans on which the towers are built. The structure presents a striking combination of American and Russian bridge practice. The lift span and towers, originally designed in America, were later revised by the Russian manufacturers to suit the limitations of their shops. The approach spans were designed in Russia.

Interest attaches to the erection of the bridge more through the attendant conditions than through any unusual features of the structure itself. The primitive nature of the plant and equipment and the absence of advanced methods would have furnished, under normal conditions, a not uninteresting task. In addition to these, the war-time shortage of materials, the scarcity of labor, the incompetency of workmen, and, after the revolution, the grossly demoralizing influences of political forces, combined greatly to aggravate the situation until

the task became quite complicated. From the natures of these intangible forces, their effects are not directly discernible in a description of the methods used. By way of preface a few examples may be cited to suggest the ramifications of these conditions.

Timber for falsework and staging was poor and small. The largest was 6 by 8 in. obtained by special arrangement for use as guard rails on the railway deck. Wherever possible, round sticks of white pine up to 30 ft. in length were used. These were rough hewn into square timbers for sills and caps.

Hardware was scarce and difficult to obtain. In one instance, a mass of correspondence and four weeks' time supplemented by considerable political pressure, were required to get four kegs of 60d. wire nails.

Even the simplest iron fittings for cranes and erecting devices were not to be obtained. Recourse was had to the bridge blacksmith who fashioned, in a most indifferent manner, essential parts for light equipment from available scrap iron.

Wire rope, ¾ in. in diameter, of hoisting line quality, was unobtainable, so that rope of standing line quality was used in its place. This rope was made up of seven strands of seven wires, each wire 1.6 mm. in diameter and was so stiff as to be almost unmanageable. Manila rope could not be secured; a weak, unreliable sisal rope was used as a substitute.

Labor was a mixed lot—generally illiterate and undependable. The foreman of machinery erection was a fisherman and boat-rigger; the ground work was done almost entirely by peasant women who also operated many of the hand winches; the men, with the exception of a few railroad employees holding special military exemption, were too old or unfit for military service.

Following the revolution and the establishment of the Soviet system of government, three workmen who had been discharged took their cases before the local Soviet which held absolute political power over all affairs. This body, after considerable deliberation, appointed a committee to see that the men were re-instated or else that the writer be forced to leave the country as having acted contrary to the best interests of the workers. This committee soon learned that the writer preferred the latter course and also that such a course involved, due to the position of the incompleted bridge, a virtual blockade of the harbor on the opening of the navigation season. An immediate reconsideration of the case resulted in an offer from the Soviet to do all in its power to facilitate the completion of the bridge and removed for the remainder of the work any misapprehension that may have existed regarding the status of the workers.

These and numerous other similar cases which might be cited, convey some idea of the incidental difficulties that were met. In themselves, each appears to be insignificant, but, in the writer's opinion, it is in surmounting these lesser and unforeseen difficulties that the constructing engineer is called on to utilize his resourcefulness and in which he finds the greatest fascination of his work.

The erection of the structural steel was carried out by methods differing from American practice principally in the nature of equipment used, the method of tower erection, and the sizes of individual pieces handled, while the method of floating in the lift span is uncommon. The arch spans were erected on pile falsework by a light steel overhead gantry crane carried on the falsework outside the trusses. No steam or electric hoists were used for the operation of this crane.

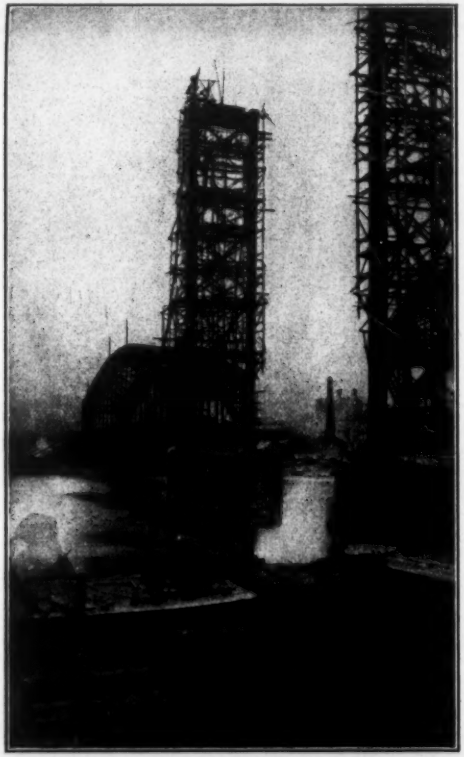


FIG. 22.—DON RIVER BRIDGE: FALSE-WORK USED FOR TOWER ERECTION. FRONT REMOVED FOR ENTRANCE OF LIFT SPAN.

FIG. 23.—DON RIVER BRIDGE SHOWING INSTALLATION OF MACHINERY AND COUNTERWEIGHTS.

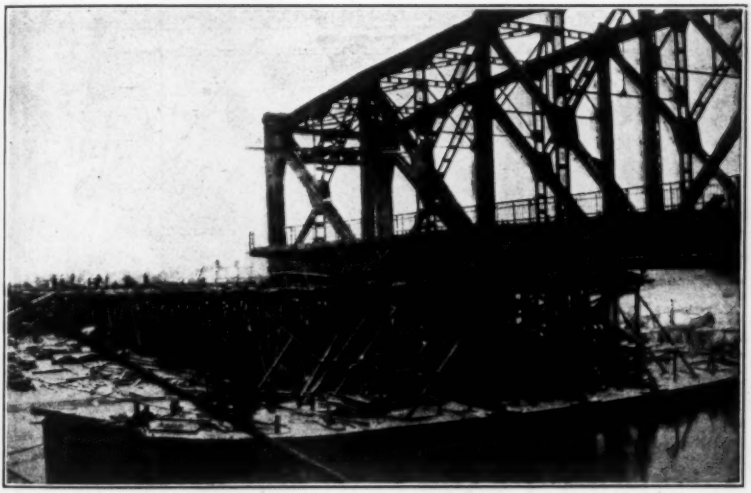


FIG. 24.—DON RIVER BRIDGE; LAUNCHING OF LIFT SPAN FROM FALSEWORK TO BARGES.

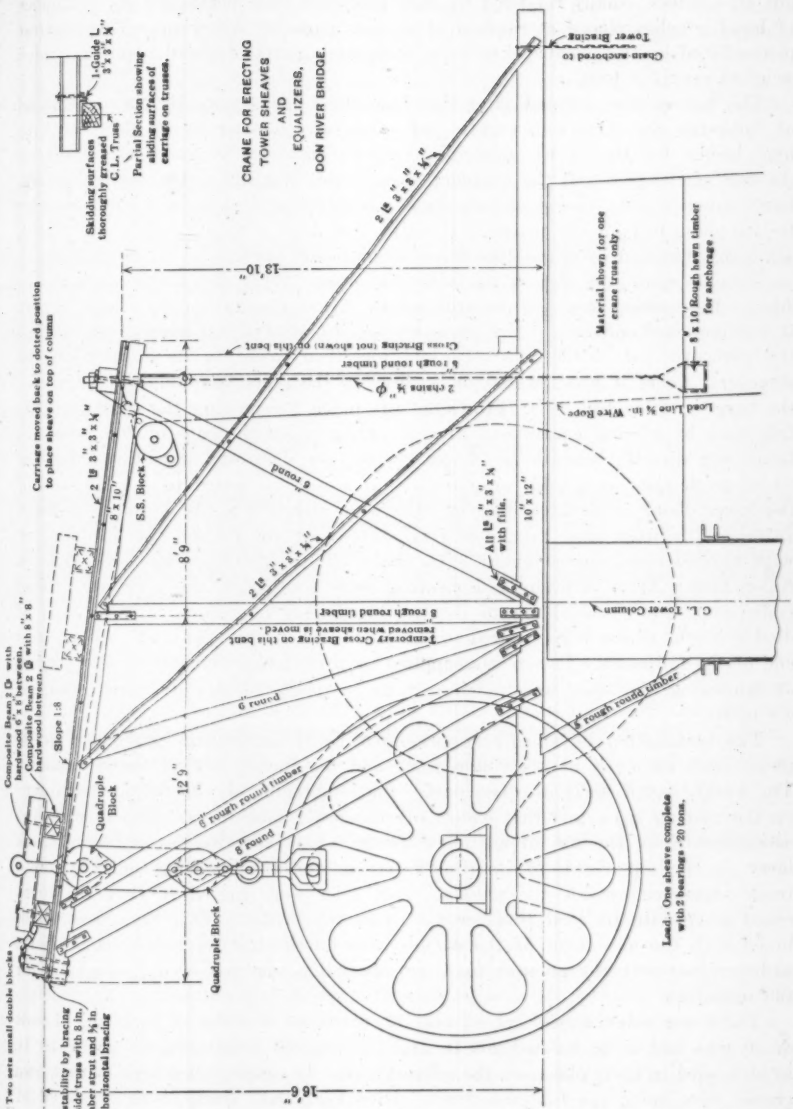

CRANE FOR ERECTING
TOWER SHEAVES
AND
EQUALIZERS,
DON RIVER BRIDGE.
Skidding surfaces thoroughly greased C. L. Truss
1-Guide L 3"x3"x¼"
Partial Section showing sliding surfaces of carriage on trusses.
Chain anchored to Tower Bracing
Material shown for one crane truss only.
5"x10" Rough hewn timber for anchorage.
Load Line ¾ in. Wire Rope
Cross Bracing (not shown) on this bent
6" rough round timber
2 chains ¾" φ
2 Ls 3"x3"x¼"
13'10"
8'9"
12'9"
16'6"
10"x12"
8"x10"
S.S. Block
6" round
All Ls 3"x3"x¼" with fills.
C. L. Tower Column
8" rough round timber
Temporary Cross Bracing on this bent removed when sheave is moved.
Carriage moved back to dotted position to place sheave on top of column
Composite Beam 2 Ls with hardwood 6"x8" between.
Composite Beam 2 Ls with 8"x8" hardwood between.
Slope 1:8
Quadruple Block
Two sets small double blocks
Provide lateral stability by bracing this point of inside truss with 8 in. rough round timber strut and ¾ in chain tie to top horizontal bracing of tower.
Load. One sheave complete with 2 bearings - 20 tons.

FIG. 25.

but all services usually rendered by such machines were performed by a number of hand winches placed at random at various parts of the crane. The heaviest pieces lifted by this means were some component parts of chord members which weighed nearly 3 tons.

The towers were erected after the supporting spans were in place. Instead of following the American practice of using gin-poles or guyed derricks with long booms for this work, falsework entirely enclosing a tower was built on the top of the pier and the adjacent span. (See Fig. 22.) On this falsework, hand winches were placed and erection was accomplished in a manner similar to that described for the trusses. Riveting was done with pneumatic equipment, air being supplied by a gasoline-driven compressor plant.

The lift span was erected on falsework about 500 ft. down stream from the site by the method used for the arch spans. After the navigation season closed, it was launched endwise on two properly rigged barges placed successively against the outboard end of the falsework and was then towed to its position in the structure, where it was lowered to bearing on the piers by pumping water into the barges. (See Fig. 24.) Provision was made for moving the span from its falsework by placing continuous lines of stringers on the caps of the falsework bents and directly beneath the lower chords; on these stringers three lines of 60-lb. track rails were laid; similarly, short ties were bolted to the bottoms of the lower chords and three lines of rails were attached to the bottoms of these. Interposed between these two lines of rails were cast-iron rollers, 8 in. in diameter, with flanged ends, spaced about 3 ft. apart. Thus, the span was erected on a roller nest. After erection and riveting were completed, the span was rolled endwise off its falsework on to the waiting barges by means of three sets of double blocks which were secured to the shore end of the span and the outboard end of the falsework. Power was supplied by three hand winches. The operation of launching the span and putting it in its place in the structure occupied six hours.

The installation of machinery presented many interesting problems due to the weights of many of the single parts and the inadequacy of the equipment. The work divided itself into two parts—that embracing the operating machinery on the moving span, and that embracing the tower machinery. The former was accomplished by the use of a timber gantry crane carrying an ordinary track lorry on the top chords of its timber trusses and moving longitudinally on a track supported outside the trusses. Fall lines were rigged so that the lorry could move with its load transversely across the bridge. This movement, combined with the movement of the whole crane longitudinally, permitted all parts to be erected with the greatest facility. Manual power was used throughout for the operation.

The tower machinery, Figs. 23 and 25, presented a different problem. Loads of 20 tons had to be hoisted 200 ft. and then moved longitudinally about 10 ft. to be landed in their places on the column tops. In order to accomplish this, two cranes were built, one for each tower. (See Fig. 25.) Each crane consisted of two wooden trusses spaced so as to permit a sheave with its bearings to pass between them, and was erected over the position a sheave was to occupy. The trusses cantilevered out in front of the tower columns sufficiently to permit the

sheave to pass the front tower bracing during hoisting; the tops of the trusses sloped from front to back 1 ft. vertically in 8 ft. horizontally and supported a carriage which held the stationary block of a set of quadruple fall blocks. The load line was led from the upper block on the carriage over a single block attached to the crane frame on the rear, thence downward to the deck of the bridge. In operation, the load was first lifted the full height with the carriage at the extreme front of the trusses, then by slacking off some safety lines made fast to prevent the carriage from moving during hoisting, the load was moved back to the desired position through the combination of the pull in the load line and the tendency of the carriage to slide down the inclined top chords of the trusses. A few slight blows with a 6-lb. sledge were ample to start the carriage back. The operating power of this crane was originally planned to be manual, but a single-drum, fixed-boom, dock crane was obtained and remodeled to serve as a hoisting engine. The best time made in hoisting and placing a sheave was 2½ hours.

The placing of ropes, construction of counterweights, and other incidental work were accomplished by means comparable to those described.

E. G. WALKER,* M. AM. SOC. C. E. (by letter).—This paper brings out a number of advantages which are likely to cause the vertical lift bridge to be developed in the future to the supersession of a great many of the existing types of movable bridges. There is no doubt that for cases in which opening spans have to be provided on rivers the regime of which varies from time to time, the advantage must lie in general with the type of bridge that can be most cheaply modified to suit the changing conditions. It does not seem to be a difficult matter to arrange a vertical lift bridge in order that conditions of variations of channel can be met.

The lift bridge type possesses one very great advantage over the swing bridge which does not seem to have been mentioned in the paper, namely, the increased safety which it possesses against accidental damage from passing vessels. In channels in which there is fairly heavy traffic this is liable to become a relatively important matter. The swing bridge, turning as it does in a horizontal plane, is at all points in its movement, except only when it is completely over its protecting pier, in danger of damage from a vessel not fully under control, whereas the vertical lift bridge at a quite early part of its movement will be lifted to such a height that collision will almost inevitably lead to the carrying away of masts, funnels, or superstructure of the ship, with the probability of relatively light damage to the bridge.

It is obvious that as the length of the span increases the economy of the vertical lift over either the swing or bascule type will be more marked. The feature of the arrangement of the machinery on which the ultimate efficiency of the bridge is most likely to depend appears to the writer to be the wire ropes, and unless the arrangement of the ropes is very carefully dealt with in the initial design of the bridge a great deal of avoidable trouble may easily be caused. The author's practice, as stated on page 583, is to use a sheave from sixty to eighty times the diameter of the rope, and provided this practice is

* London, England.

followed no great trouble should be experienced through the wearing out of ropes of good standard manufacture. When large ropes are required, rigid adherence to a specification like this will often necessitate the provision of a large sheave diameter and in such cases there sometimes exists a tendency to cut down the sizes of pulleys and drums on the score of economy.

The writer knows of instances of swing bridges operated by wire ropes in which, with no very abnormal traffic requirements, ropes 7 in. in circumference have had to be renewed about every six months, and he believes that the principal cause of the trouble has been the failure of the original designers of the machinery to appreciate the necessity for keeping the diameters of the sheaves large in order to reduce the range of stress in the individual wires and strands forming the rope, which is the principal reason for the fatigue failure of cables when run over pulleys of small diameter.

The three bridges which the author has so well described and illustrated are all purely utilitarian designs, but the writer sees no reason why this type of movable bridge cannot be adapted to locations requiring more ornamental treatment. In fact, it seems likely that it would be much easier to design such a bridge on this principle than on any other principle that is used for movable bridges. It would be a relatively simple matter to apply appropriate architectural treatment to the towers of the moving spans, and the fixed spans could be built either as arches, girders, or any other form of structure that is adapted to ornamental treatment. Preferably, in such cases, the lifting span could be treated in such a way as to harmonize with the fixed spans and the whole arranged to form a satisfactory architectural unit. Such treatment is very difficult in cases of bascule bridges and practically impossible for swing bridges. It would seem therefore that there is a future for this type of bridge in over-congested waterways, in cities, and locations where ornamental effect is required.

HORATIO P. VAN CLEVE,* M. AM. SOC. C. E. (by letter).—Mr. Howard's illuminating paper bears witness to his thorough knowledge of, and belief in, his subject, and those who heard him present it were impressed by the simplicity of the vertical lift bridge as well as by its adaptability to a wide range of conditions.

Passing reference was made by the author to an accident to one of the lift bridges in which a boat, through a mistake of signals, collided with the bottom chord, bending it considerably out of line, but without serious consequences to the structure. Fig. 26 is a view of this injured chord, showing the immediate results of the collision. It will be noted that, incidental to the deflection of 1 ft. the lacing bars are badly bent and several rivets have been sheared off. This is a large and important railway bridge carrying fast passenger traffic which was not interrupted by the accident. Fundamentally, the lift bridge is a simple span and, consequently, the bottom chord is always a tension member. If a blow of like magnitude had occurred against the compression chord of a bascule or swing span, the sequel might have been quite different.

Aside from the points of interest in the lift bridge as a structure, it has claims to distinction as a machine and although the author touched on some

* Cranford, N. J.

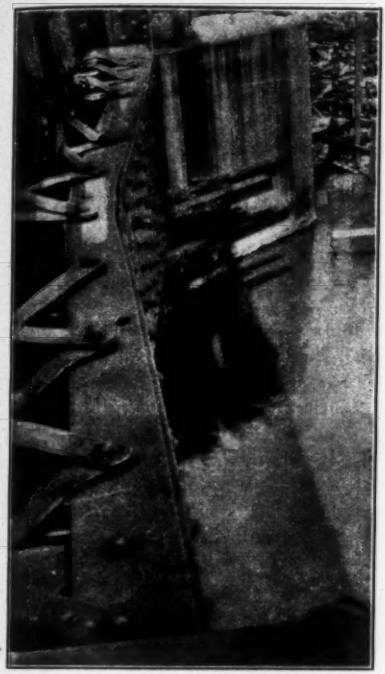


FIG. 26.—VIEW OF INJURED BOTTOM CHORD OF BRIDGE AFTER COLLISON.

FIG. 27.—ATTACHMENT OF SIXTEEN 2¼-IN. ROPES AT CORNER OF BRIDGE.

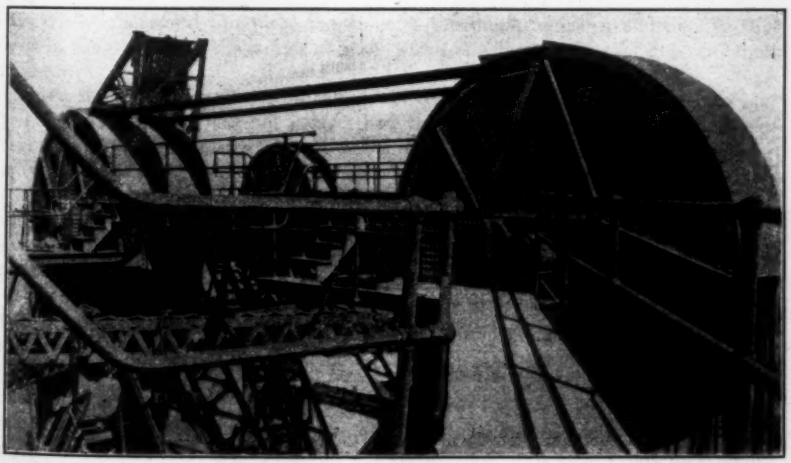


FIG. 28.—TWO SETS OF SUPPORTING SHEAVES ON BRIDGES ON PENNSYLVANIA LINE IN SOUTH CHICAGO, ILL.

elementary mechanical features, further reference and illustration may not be amiss.

Supporting Features.—In his comments on the qualities of wire rope, the author refers to the use of forged steel sockets. The old bridge at Halsted Street, Chicago, Ill., is attached to its supporting cables by means of a large number of cable clamps on each rope. This arrangement is crude, but was the best obtainable 27 years ago. It may be of interest to note that drop forged sockets of the sizes needed for large lift bridges were not made prior to the demand created by lift-bridge requirements.

Fig. 27 shows the attachment of sixteen 2¼ in. ropes at one corner of a large bridge over the Chicago River. The compact arrangement shown would not have been possible before the days of the drop forged socket for this size of rope. The load supported at this point is 400 tons.

Mr. Howard has referred to two types of supporting sheave, the steel casting and the form built of rolled steel plates and shapes carrying cast-steel rim sections. The construction of the latter is illustrated by Fig. 28, showing parts of two sets of such sheaves on two bridges for the Pennsylvania Lines in South Chicago. Two wheels only of the near span are visible, the other two forming a portion of the distant bridge. In the background is visible the end of a large bascule bridge standing open. These sheaves are of 15 ft. 0 in. pitch diameter. Their centers are composed of four semi-circular plates, spliced in pairs to form two disks which are held solidly together by fourteen diaphragms. The cast-steel rims, which are built in sections, were originally designed to transfer their loads to the disks through bearing on the milled edges of the latter, but before these sheaves were complete, experience with similar sheaves on the Willamette River Bridge of the Oregon-Washington Railroad and Navigation Company demonstrated that the degree of accuracy in machine work required for such transfer of stress was not practicable of attainment, and, therefore, splice plates were added so as thoroughly to rivet the rims to the diaphragms in addition to the rivets in the disks. At the center of each wheel is a large cast-steel hub carrying the shaft.

After these bridges were put into service, it developed that the fit between the hubs and the disks was not perfect and that movement was going on between them. This was corrected by driving tool-steel pins in the joints between hubs and disk plates, and parallel to the axis of the shaft. This proved to be entirely successful. A full explanation with illustrations of the method used in this work is given in a paper entitled "Mechanical Features of the Vertical Lift Bridge", presented by the writer before the American Society of Mechanical Engineers in December, 1918.*

Counterweights have been attached to the suspending ropes by several forms of equalizers. Fig. 29 shows the arrangement used for connecting sixteen ropes to two counterweight units. Below the lowest visible heart-shaped bars are two others of the same general form, to each of which a pin on the counterweight section delivers its load. The ears on the top bars are for use in case of rope replacement. Since this photograph was taken, a large clamp has been put on each set of ropes immediately above the equalizer unit to prevent movement

* *Transactions*, Am. Soc. Mech. Engrs., Vol. 40 (1918), p. 1037.

and resulting wear on the pins. Wells were provided in these counterweights for adjustment of weight, and although balance has been effected, it will be noted that two pockets are visible, which are still unfilled. Drain holes are provided at the bottom of these wells.

Guiding and Centering.—To prevent any possible binding during operation it is considered advisable to allow plenty of clearance between the span and counterweight guides and the tracks on which they run, and this means that near the bottom of the run closer centering must be introduced in cases where railroad tracks cross the structure. In some cases, where bridge locks are used, centering has been accomplished as a feature of these locks. This is the case for the lifting decks of the bridges over the Missouri and Willamette Rivers described in Parts III and IV of the paper. In most other cases separate centering castings have been used.

Fig. 30 shows one of these castings located in a tower column directly above the lift-span shoe at the expansion end. The mating casting on the lift span carries a projection which is wedge-shaped at its lower end, so that the two sets at this expansion end of the span reduce the lateral clearance to the required limit. At the other end of the lift span the projections on the span castings have bulb ends fitting into sockets in the tower castings, thus restricting longitudinal as well as lateral movement, and taking the train thrust.

The writer has been Resident Engineer on the erection of seven of these bridges, all of them for railroad traffic, and he feels that for this class of service separate centering devices, because they are susceptible of independent adjustment, are better than the flaring of the guide tracks mentioned in the paper.

Fig. 30 also shows the connection of the lowering ropes to the tower, and the turnbuckles for adjustment. Two ropes were used in this case on account of the heavy loads involved, and to reduce the bending stresses.

Fig. 31 shows a typical arrangement of guiding and centering devices as used on most of the bridges thus far constructed.

Locks.—In Part I of the paper, the statement is made that locks are not necessary, although occasionally used as part of a track-interlocking system. Such a system of track locks is illustrated by Fig. 32, which shows a portion of the floor of one of the bridges of the Pennsylvania Lines over the Calumet River in South Chicago. Across the center of the photograph lies the joint between the lift span and one of the fixed spans. Between the ribbed castings and the rails lie the standard joint bars of the Railroad Company. These are manganese-steel castings fitting closely the rail webs and operated by links connected through a set of cranks and shafting to a worm drive and electric motor. From this description it will be realized that the end of the span is securely locked down although this locking is incidental only, the main object of the joint bars being to insure that the rails are in perfect alignment and to carry the wheels without jar across the joints. That this latter object is satisfactorily accomplished is evidenced by the fact that fast passenger trains cross this bridge at frequent intervals with no appreciable decrease in speed, and with no more jar than is caused by ordinary rail joints.

This apparatus is electrically interlocked with the span operating mechanism so that the joint bars cannot be driven until the span is fully seated, and so

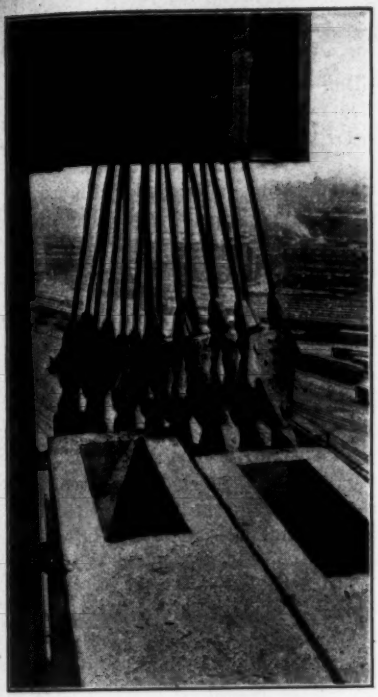


FIG. 29.—ARRANGEMENT FOR CONNECTION OF ROPES TO COUNTERWEIGHT UNITS.

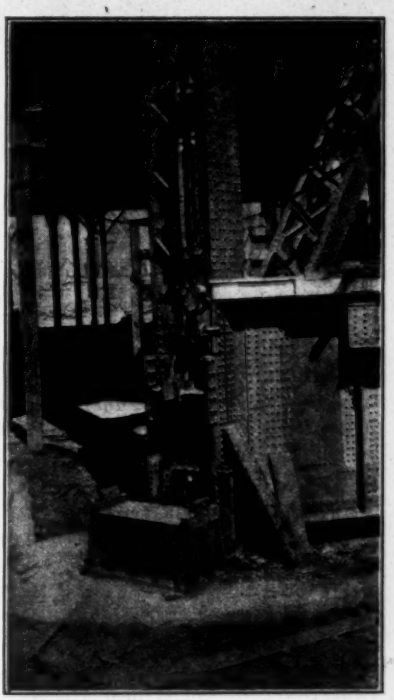


FIG. 30.—VIEW OF CENTERING CASTING IN TOWER COLUMN.

FIG. 31.—TYPICAL ARRANGEMENT OF GUIDING AND CENTERING DEVICES.

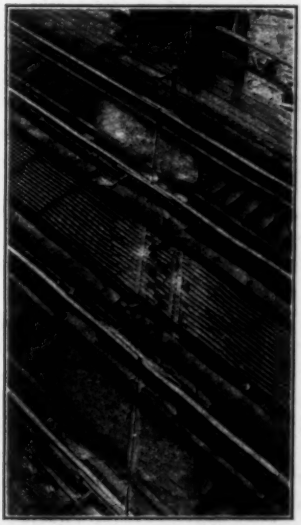


FIG. 32.—SYSTEM OF TRACK LOCKS ON BRIDGE.

that the span cannot be raised until the bars are drawn. At the rear end of each bar a pin, controlled by a bell-crank, passes through the bar, the joint casting, and the rail web, effectively locking the bar against movement during the operation of trains.

Running diagonally across the center of the photograph (Fig. 32) are the fourteen lines of pipe of the signal system. These automatically unjoint as the span is lifted, connecting again when it returns to its seat. As the floor is horizontal at all times no shifting of the lengths on the lift span occurs. Carrying such a system across either a bascule or swing bridge would be attended with difficulties.

Operating Equipment.—For a given clear waterway, the operating machinery required for a lift span is considerably less than for either the bascule or the swing span largely because wind is such an important element in the design of the last two, while in the first the wind effect is merely to increase the friction of guide rollers, adding comparatively small resistance to operation. It follows as a corollary that the lift span, practically independent of the wind, may be operated safely at times when the movement of other types would be fraught with considerable danger. Although it may be argued that there is little weight to this claim because operation is seldom required in times of high wind, it is nevertheless true that heavy gusts do occur in times of moderately light winds, and if either of the bascule or swing type of span is partly open, delay if not damage may result. The lift span, on the other hand, is as perfectly under control in times of high wind as when the wind is not blowing.

The mass to be moved in the case of the lift span is less than for either of the others and, consequently, friction and inertia are less. Because the forces to be overcome are less for the lift span than for other movable types, it follows that operating machinery will be less, and this assertion is borne out by the comparison of operating costs of the five bridges over the Willamette River referred to by the author. Fig. 3 is an excellent view of the operating train required for one of the heaviest and latest bridges constructed. It will be noted that the electric motors are not yet in place. The small gas engine for auxiliary operation looks insignificant in comparison with the rope drums, but should be able to make a full operation in about two hours.

For further points of interest in the design and construction of the vertical lift bridge the writer would refer to his paper "Mechanical Features of the Vertical Lift Bridge" already mentioned. It gives his experience in connection with the design and erection of these structures during the period of their greatest development, from 1909 to 1914.

F. W. GREEN,* M. AM. SOC. C. E. (by letter).—The author is to be congratulated on the very thorough and effective manner in which he has covered the subject of vertical lift bridges. In recent years this type of bridge has attained a popularity to which its many excellencies justly entitle it. Although the writer has had no experience in the design of swing spans moving in a horizontal plane, his contact with engineers who have had such experience, convinces him that the swing span for heavy live loads is destined to enjoy a much less degree of popularity than it did prior to the advent of the vertical lift type of span.

* St. Louis, Mo.

The adaptability of this type for crossings of alluvial rivers is an important asset. In such streams the normal location of the channel is adjacent to one bank or the other, shifting sometimes from one side of the river to the other in a relatively short space of time, and sometimes in a surprising manner. The writer knows of several instances where additional draw spans had to be constructed in such circumstances. With the vertical lift type, as stated by the author, the problem is relatively simple, in that towers and machinery may be removed from one location and re-erected in another. Two bridges with this type of span listed in Table 1, namely, those over the Black River and over the Little River near Jonesville, La., were erected in 1912-13, at which time the writer was in charge of the construction for the Louisiana and Arkansas Railway Company. These structures were very satisfactory, and have never given any trouble of any kind, either in connection with their operation or maintenance. It was found that this type of structure was popular with the Army engineers, as well as with parties navigating steamboats, because of the greater facility for maneuvering the boats through the draw in times of swift current, due to the large opening.

The writer believes that if wire rope conforming to appropriate specifications is provided for service of this character in the first instance, and thereafter well maintained by the application of a good quality of cable dressing, it will last indefinitely. It is important that provision be made to equalize the load on all ropes carrying counterweights. This was done in a very ingenious manner in the bridges over the Black River and the Little River.

The erection of the vertical lift type offers no particular difficulties. The vertical lift span, when designed for the dead, live, impact, and wind loads, requires no additional provision for other stresses when in the open position. This is an important advantage over the horizontal swing spans.

VICTOR H. COCHRANE,* M. AM. SOC. C. E. (by letter).—The paper describes in detail three important bridges, each containing a different type of vertical lift span. The simplicity of the vertical lift bridge in its elements and its adaptability to various conditions are clearly brought out. In view of the title, however, the paper is somewhat disappointing in that so much space is devoted to details which have nothing to do with vertical lift spans as such, and so little to the discussion of those questions of design and cost which are of vital interest in comparing the vertical lift with other types of movable bridges. Several important matters are dismissed with only a sentence or two.

The author states in his Synopsis that other suspending elements than wire ropes "have been considered and even tried" and that Table 1 contains general data concerning "the more important lift bridges that have been built." These two statements taken together might lead the uninformed reader to conclude that no vertical lift spans of consequence other than those listed in Table 1 have been built. Table 6 gives data which the writer has at hand concerning a few spans which are longer and heavier than some of those listed in Table 1. Perhaps the author hesitated to mention bridges about which he had no other information than that contained in descriptions in the technical press; Table 1 is in reality a list of cable lift bridges, and to be complete should include those spans suspended by other means. Some of these will be described briefly by the writer.

* Tulsa, Okla.

TABLE 6.—LIFT BRIDGE DATA.

Bridge* over	Kind of traffic.	Length of span, in feet.	Maximum clear height above high water, in feet.	Normal height of movement, in feet.	Total moving load span, counterweights, etc., in tons (approximate).
Arkansas River, near Pine Bluff, Ark., for Jefferson County Bridge District	Single-track railway and highway	239	60	49	1 250
Fraser River, at Fort George, B. C., Canada	Single-track railway and highway	106	....	40	420
Louisville and Portland Canal, for Pennsylvania Lines West of Pittsburgh	Double-track railway	360	55	32.4	3 200
Louisville and Portland Canal, for U. S. Government	Highway	210	55	40	365
Rideau Canal, for the City of Ottawa, Ont., Canada	Double-track electric railway and highway	106	80	20	570
Steilcoom Creek, Tacoma, Wash., for Northern Pacific Railway	Double-track railway	96	....	48½	545

* The Pine Bluff Bridge was designed by Messrs. Hedrick and Cochrane; the Louisville and Portland Canal Bridge for the Pennsylvania Lines, by Waddell and Son; and the others by the Strauss Bascule Bridge Company.

The characteristic feature of the movable spans described by the author is the use of wire ropes both for suspension and operation. These ropes are generally made of plow steel and consist of 6 strands of 19 wires each, laid up around a center of oil-saturated hemp. The diameter of the individual wires is about one-fifteenth of the diameter of the rope. Sockets are used to attach the ropes to the span and counterweight. Whether warranted or not, there has unquestionably existed among engineers a feeling of doubt as to the entire reliability of wire ropes and sockets, which may explain, in part at least, why the vertical lift type did not sooner come into use, and why it has not been utilized more extensively. It is true, as the author intimates, that wire ropes are reliable, efficient, and widely used, but it must be remembered that in the case of many uses of wire rope a failure does not result in more than temporary inconvenience and delay, while, in the case of a lift bridge, failure might be disastrous. For many purposes there is no satisfactory substitute for wire ropes, but this is not necessarily true of the vertical lift bridge. Consequently, the question of the use of wire ropes should be carefully considered in all its aspects.

The individual wires of a rope are subjected to bending stresses when passing over the sheave. The author says that the "extreme fiber stresses from bends of large radius are very small", but in lift bridge practice the bends are not of very large radius and the extreme fiber stresses can hardly be considered to be very small.

There seems to be a wide difference of opinion among various authorities in regard to the amount of bending stress developed in the individual wires as the rope runs over a sheave.

Let d = diameter of rope, in inches;

D = diameter of sheave, in inches (on rope center);

r = ratio $\frac{D}{d}$;

d' = diameter of individual wire, in inches;

f = extreme fiber stress due to bending.

In the case of straight wires of diameter, d', running over a sheave of pitch diameter, D, it can be shown that

$$f = \frac{E\,d'}{D}$$

Reuleaux, Rankine, and Unwin assume that the extreme fiber stresses in a wire rope are the same as for a single straight wire. For wire ropes containing 6 strands of 19 wires each,

$$d' = 0.067\ d, \text{ approximately.}$$

Hence, under the previous assumption

$$f = \frac{0.067\ E}{r},$$

if

$$E = 28\,500\,000,$$

$$f = \frac{1\,910\,000}{r}.$$

Other authorities take into account the lay of the rope and other factors. Hewitt's formula, specified by Waddell, gives values of f approximately equivalent to $\frac{1\,840\,000}{r}$.

Leffler's formula* is equivalent, for 6 by 19 ropes, to

$$f = \frac{1\,800\,000}{r}.$$

In 1908, R. W. Chapman developed a formula equivalent to

$$f = \frac{1\,500\,000}{r},$$

Josef Hrabak's formula† amounts to

$$f = \frac{840\,000}{r}.$$

In 1913, the American Steel and Wire Company published tables of bending stresses in accordance with the formula

$$f = \frac{800\,000}{r},$$

based on actual tests which showed that the modulus of elasticity for a twisted wire rope with a hemp center does not exceed 12 000 000. Tests made by the U. S. Bureau of Standards‡ show that the modulus of elasticity for new ropes is probably below 8 000 000. It appears, however, that in all these tests the modulus of elasticity, so-called, was determined by testing the rope in tension. As pointed out by Shortridge Hardesty,§ Assoc. M. Am. Soc. C. E., the assump-

* "Specifications for Movable Railroad Bridges," *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 386.

† "Die Drahtseile", 1902.

‡ *Technologic Paper No. 121*, p. 56.

§ *Transactions*, Am. Soc. M. E., Vol. 40 (1918), p. 1077.

tion that the modulus of elasticity for stretch may be used for computing bending stresses is by no means warranted. It can be readily seen that, since an individual wire in a rope is in the shape of a compound helix (except the center wire of a strand, which is a simple helix), the additional length of the twisted wire over that of a straight wire will slightly reduce the bending stresses as compared with those in a straight wire. It appears further that in those parts of the bent wire parallel or nearly parallel to the sheave, shearing stresses will be set up as in a coil spring. The deflection due to shear is about 25% greater than that due to transverse flexure, hence the shearing stresses should relieve the bending stresses to some extent. The greater the twist the greater will be the distortion due to shear. Mr. Hardesty shows in the discussion previously referred to that the theoretical formula agrees with some actual bending tests made by Mr. Chapman.*

It is evident, too, that even if the modulus of elasticity as determined by tension tests on new rope is applicable to the determination of bending stresses therein, it would not be safe to use this modulus for an old rope long exposed to the weather. It is practically certain that even the best of lubricants, carefully applied at frequent intervals, will not keep the rope as pliable as when new. Certainly, a rope dressing which merely covers the surface and does not penetrate all the strands and saturate the core, will not keep the rope in good condition.

In view of these considerations, and pending a fuller investigation of the whole subject, the conservative designer will use values of bending stresses as determined by some formula such as that of Leffler or Hewitt.

In the following discussion, the Leffler formula is compared with the formula of the American Steel and Wire Company; these may be termed for convenience of reference the theoretical and the empirical formulas, respectively. The writer understands that the author's designs are made according to the Hewitt formula, which gives about the same results as that of Leffler.

The author states on page 583 that for suspending cables values of r of from 60 to 80 afford satisfactory proportions between direct tension and bending stresses. For $r = 60$, $f = 30\,000$ lb. per sq. in. by the theoretical formula and 13 300 lb. per sq. in. by the empirical formula. For $r = 80$ the stresses are 22 500 and 10 000 lb. per sq. in., respectively. These stresses can hardly be called "very small." The allowable unit stress for direct load is equal to the maximum unit tension less the extreme fiber stress due to bending. Leffler† specifies that the maximum unit tension shall not exceed one-sixth of the ultimate strength, which for plow steel cables, 1½ in. in diameter and more, may be taken as 192 000 lb. per sq. in., giving a working stress of 32 000 lb. per sq. in. Waddell‡ specifies that the direct stress (not including bending) shall not exceed one-sixth of the ultimate strength, and that the maximum unit tension shall not exceed one-half of the elastic limit. For plow steel ropes of the sizes generally used these limits are approximately 32 000 and 57 000 lb. per sq. in., respectively. The actual values vary somewhat for ropes of different sizes, as shown in the standard strength tables.

In the case of the Columbia River Bridge (Part II), the data and stresses

* Described in his paper published in *Engineering Review*, October, 1908.
† *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 337.
‡ "Bridge Engineering", p. 1712.

are approximately as follows: Suspended load, 1 200 tons; 64 suspending cables, 2 in. in diameter; suspending sheave, 144 in. in diameter; $r = 72$.

	By the theoretical formula:	By the empirical formula:
Bending stress	25 000	11 100
Direct stress $= \frac{2\ 400\ 000}{64 \times 1.613}$	23 200	23 200
Total stress	48 200	34 300

The upper deck of the Harriman Bridge (Part IV) weighs about 1 700 tons and is supported by 64 cables, $2\frac{1}{4}$ in. in diameter; the sheaves are 14 ft. in diameter, and the ratio, r, is 75, hence

	By the theoretical formula:	By the empirical formula:
Bending stress	24 000	10 700
Direct stress $= \frac{3\ 400\ 000}{64 \times 2.075}$	25 600	25 600
Total stress	49 600	36 300

In these two cases the bending stress by the theoretical formula is approximately the same as the direct stress, which is good practice. The theoretical stresses are much higher than one-sixth of the ultimate strength, as specified by Leffler; indeed, it would be rather difficult in many cases to keep the maximum unit stress down to about 32 000 lb. per sq. in. The designer is confronted with the difficulty that if he increases the diameter of the ropes in order to cut down the direct stress, the bending stress is increased so much that little or nothing is gained. The only alternative is to use very large sheaves or a large number of ropes, both of which lead to difficulties in design and construction.

This situation naturally results in the use of higher unit stresses than otherwise might be applied. The writer is of the opinion that the ratio of sheave diameter to rope diameter as given by the author is too small and that the minimum ratio should be not less than 80. The ratio for the Halsted Street Bridge, which is cited as an example of long-lived ropes, is 96, and the maximum unit tension is less than 40 000 lb. per sq. in., using the theoretical formula for bending stresses, or 30 000 lb. per sq. in., according to the empirical formula.

The author states that the life of a wire rope operating over a sheave is a function of the number of operations. It would seem that the life of a rope under heavy stress and operating over a sheave of small diameter would be much less than that of a lightly stressed rope running over a large sheave. The friction and consequent wear of the small wires would be augmented by an increase in load or by a decrease in sheave diameter. The writer is inclined to agree with Leffler in limiting the maximum unit tension to one-sixth of the ultimate strength, especially if the empirical formula for bending stresses is used. If the equalizing devices are eliminated there will be no difficulty in using enough ropes to keep the stress within this limit.

In the case of the Harriman Bridge the sheaves are very large, and it may be of interest to compare the number of ropes of different sizes required and the corresponding width of drum under different assumed total stresses. Using the

theoretical formula for bending stresses and assuming a total stress of 49 600 lb. per sq. in., there results:

	Diameter of rope, in inches: 1¾	2	2¼
Number of ropes required	88	76	64
Total area, in square inches	110.3	122.6	132.8
Ratio, r	96	84	75
Bending stress	18 800	21 400	24 000
Direct stress	30 800	27 800	25 600
Total stress	49 600	49 200	49 600
Approximate drum width	3 ft. 8 in.	3 ft. 8 in.	3 ft. 6 in.

These results show that there is no particular advantage in using larger ropes except the decrease in the number of equalizers required.

If, now, one attempts to keep the total stress below one-sixth of the elastic limit, or, say, 32 000 lb. per sq. in., as specified by Leffler, it is found that about 200 ropes are required in the case of each of the three sizes previously assumed, showing a decided advantage in favor of the smaller ropes. It is evident, however, that such a large number of ropes could hardly be used, especially if equalizers are used.

Now, assuming that the bending stresses are computed by the empirical formula, and that the total extreme fiber stress shall not exceed 32 000 lb. per sq. in., the results are as follows:

	Diameter of rope, in inches: 1¾	2	2¼
Number of ropes required	116	96	80
Total area, in square inches	145.4	154.8	166.0
Ratio, r	96	84	75
Bending stress	8 300	9 500	10 700
Direct stress	23 400	22 000	20 500
Total stress	31 700	31 500	31 200
Approximate drum width	4 ft. 9 in.	4 ft. 6 in.	4 ft. 3 in.

It appears from the foregoing results that if the total stress is not to exceed one-sixth of the ultimate strength, there should have been about three times as many ropes as were actually used, based on the theoretical formula for bending stresses, and 25% more ropes if the empirical formula is used. The writer considers the author's working stresses to be too high, especially as no allowance is made for impact. In all cases the smaller ropes would be preferable were it not for the complication of the equalizers. The question of the use of equalizers will now be discussed.

The author states that "the load on a group of ropes of unequal length or uneven stretch may be distributed equally to each by equalizing bars or levers." The use of the word "may" possibly indicates that there is some doubt as to

whether equalizing devices are entirely effective or whether they should be used at all. As a matter of fact they have been used on nearly all the cable lift bridges that have been built, and are worthy of fuller description. Waddell's specifications for equalizers are as follows:*

"The counterweight ropes shall, as a rule, be connected directly to the steel-work on the lift span, but on the counterweight side they shall be attached to equalizers, which, in turn, are attached to the counterweight. Each equalizer bar shall be made with the center pin below the end pins. This distance will depend on the layout and design of the bars, because no vertical links shall be used between the bars except to change the direction, as noted below. The layout of the equalizers will depend on the type of counterweight used, whether sectional or solid. In the sectional type four ropes shall generally be attached to each section, and the equalizer bars shall be placed parallel to the axis of the bridge. In the solid counterweight, the upper equalizer bars, to each of which two ropes are attached, shall, as a rule, be placed transversely to the axis of the bridge. They shall be attached to the lower equalizer bars, which run parallel to the axis of the bridge, by criss-cross links. On the span side, open sockets shall generally be used; and on the counterweight side, closed sockets. All pins connecting the various equalizer bars shall have a head $\frac{1}{4}$ in. thick on one end and a split cotter on the other. The pin connecting the bottom bars to the counterweight hanger shall have the ends threaded for Lomas nuts. Provision shall be made for removing any pin connecting the sockets to the bars or lift span in case it becomes necessary to replace a rope, and the equalizers shall be so designed that a rope can be replaced without supporting the counterweight. All pins in the upper equalizer bars shall be of the same size, *viz.*, that used for the sockets. The clearance between any two parts shall be greater than $\frac{1}{2}$ in., which amount shall be considered no clearance."

Fig. 33 shows one of the latest designs, there being 16 ropes, 8 of which are attached to each of two independent counterweights. For a single solid counterweight still another and a very heavy bar would have been required. Half the bars to which the rope sockets are attached are long and the other half are short, as shown, in order to reduce the lead of the ropes as they pay off the drum.

Fig. 34 shows what will happen in the case of "unequal length or unequal stretch." It is evident that with the style of equalizer bar used, the load on the pair of ropes will not be equal. Suppose, for example, that the ropes are 150 ft. long; that the modulus of elasticity for stretch is 12 000 000, as stated by the American Steel and Wire Company (Waddell uses 17 000 000, but the exact value is not of particular consequence); that the inequality of length is 20% of the elastic stretch; and that the average direct stress is 25 000 lb. per sq. in. Then the stretch is

$$\frac{150 \times 12 \times 25\ 000}{12\ 000\ 000} = 3.75 \text{ in.}$$

The assumed inequality of length is 0.75 in. With the dimensions shown the equalizer bar will rotate so that the lever arms become 3.40 in. and 5.07 in., respectively. The shorter rope will carry 60% of the total load, P, and the longer rope 40%, thus making the unit stresses in the rope 30 000 and 20 000 lb. per sq. in., respectively, instead of 25 000 lb. as intended.

It does not seem reasonable, however, to suppose that there will be much difference in the modulus of elasticity for the stretch of two ropes made at the same

* "Bridge Engineering", p. 1719.

time, although there might be some slippage of the socket connection. Assuming that there is no difference in stretch for the same unit stress, any initial difference in length will be reduced when the ropes take their load.

Referring to Fig. 35 let

i = initial difference in length of ropes;
d = final difference in length of ropes;
s = total movement of lower pin;
a = distance center to center of upper pins;
h = distance of lower pin below upper pins;
x = final lever arm of left-hand pin.

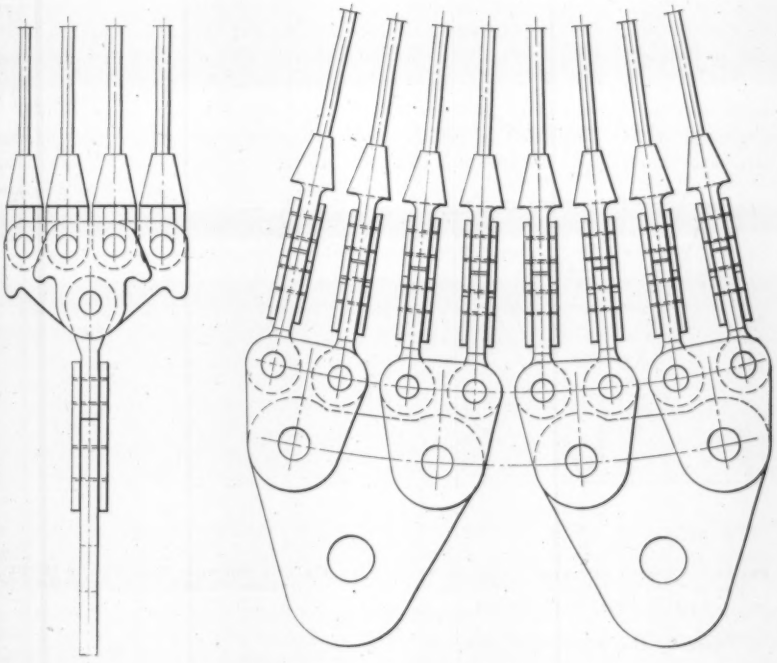


FIG. 33.—TYPICAL EQUALIZER DETAIL.

The stretch of the shorter rope is

$$s + \frac{1}{2}(i - d),$$

and the stretch of the longer rope is

$$s - \frac{1}{2}(i - d).$$

Since the load on each rope is proportional to the stretch, it follows that the load on the shorter rope is

$$\frac{P}{2} \times \frac{s + \frac{1}{2}(i - d)}{s},$$

and the load on the longer rope is

$$\frac{P}{2} \times \frac{s - \frac{1}{2}(i - d)}{s}.$$

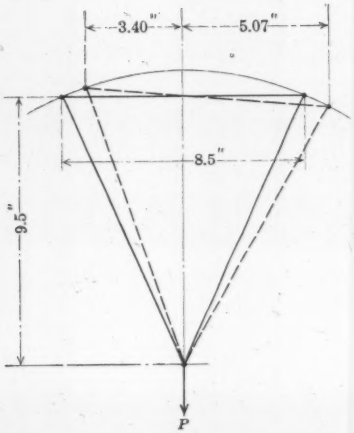


FIG. 34.—SHOWING ROTATION OF EQUALIZER BAR IN CASE OF UNEQUAL LENGTH OR UNEVEN STRETCH OF ROPES.

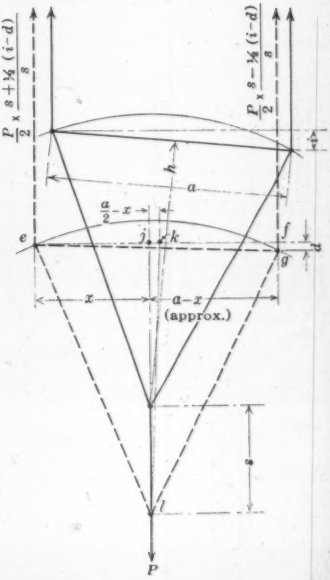


FIG. 35.—SHOWING REDUCTION IN INITIAL INEQUALITY IN LENGTH OF ROPES.

Then, by the equality of moments

$$\frac{P\,x}{2} \times \frac{s + \frac{1}{2}(i - d)}{s} = \frac{P\,(a - x)}{2} \times \frac{s - \frac{1}{2}(i - d)}{s},$$

whence

$$x = \frac{a}{2}\left(1 - \frac{i - d}{2\,s}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The triangle, *efg*, is similar to the triangle, *lkj;* therefore,

$$\frac{a}{2} - x : d = h : a,$$

hence

$$d = \frac{a}{2\,h}\,(a - 2\,x) \qquad (2)$$

Substituting this value of d in Equation (1), we get the formula,

$$x = \frac{a}{2} - \frac{a\,h\,i}{4\,h\,s + a^2} \qquad (3)$$

In the case previously assumed: $s = 3.75$ in.; $i = 0.75$ in.; $a = 8.5$ in.; $h = 9.5$ in.

From Equation (2), $d = 0.25$, being only one-third of the initial inequality; also $x = 3.97$ in. and $a - x = 4.53$ in., approximately.

The unit stresses in the short and long ropes are 26 650 and 23 350 lb. per sq. in., respectively.

These results are only roughly approximate, inasmuch as a number of factors were neglected, including the pin friction, the friction due to the slipping of the ropes on the sheave above, and the change of inclination of the ropes.

Assume, now, that no equalizing device is used, each rope being connected independently to the counterweight, and that one of the ropes is $\frac{3}{4}$ in. too short and another $\frac{3}{4}$ in. too long, making $i = \frac{3}{2}$ in., as before. The short rope will stretch 4.125 in. and the unit stress will be

$$25\,000 \times \frac{4.125}{3.75} = 27\,500 \text{ lb.}$$

This is but little greater than the stress computed for the equalizer arrangement.

Dr. Waddell* gives the following tabulation of allowable variations in fabricated lengths of wire ropes.

Manufactured length	Variations (+ or —)
0 — 100 ft.	$\frac{1}{4}$ in.
100 — 200 ft.	$\frac{3}{8}$ in.
200 — 300 ft.	$\frac{1}{2}$ in.
300 ft. and upward	$\frac{5}{8}$ in.

It will be seen that the inequality of length assumed in the cases considered above, $\frac{3}{4}$ in., agrees with the tabulation. It should be entirely practicable to attach the sockets with a smaller variation in length if desired.

Hence, it is concluded that, in so far as providing for unequal length or uneven stretch is concerned, such so-called equalizing devices are entirely useless; not only would their omission result in a considerable reduction in cost, but it would greatly simplify the details of rope attachment. The working stresses could be reduced to any desired amount. The towers would not have to be so high, thus effecting a saving in structural steel.

It may be objected that if the equalizers are omitted it will be more difficult to remove and replace the ropes, but such does not appear to be the case. Apparently, the lugs or ears shown in Fig. 33 are intended as seats for jacks, but the stress in a rope connected to one of the shorter bars cannot be relieved by jacking, since there is not enough vertical play to take out the stretch. These lugs can be

* "Bridge Engineering", p. 1719.

used as seats for a clamping device designed to stretch the ropes, but obviously a similar method of removing the ropes can be applied if no equalizers are used.

Not only do these equalizers appear to be useless, but, in the writer's opinion, they are open to the more serious objection that they are unsafe. It must be admitted that there is a possibility of the failure of a rope, either by the breaking or slipping of a socket or the breaking of the rope itself, especially when such high working stresses are used. Horatio P. Van Cleve, M. Am. Soc. C. E.,* states, in connection with an equalizer set similar to that shown in Fig. 33, but designed for 16 ropes, that if one rope of a pair should break, "the load would shift laterally and readjust itself to such an extent that the remaining 15 ropes would each get an approximately equal share of the load originally supported by the broken one."

The writer is unable to check this statement. For the eight-rope equalizer set shown in Fig. 33, if one of the ropes connected to a short bar should break, the stress in the other will be increased about 40%, assuming a normal stretch of 3.75 in., as in the case previously discussed. Failure of one of the ropes connected to a long bar would entirely relieve the stress in the other, and the stress in the next pair of ropes would be increased about 67 per cent. Furthermore, it is to be expected that these adjustments will take place with some shock or jar, and it is not improbable that the direct stress will in some cases be nearly double the original stress. Thus, the failure of one rope might lead to the progressive failure of the entire set.

The first equalizers used consisted of straight bars connected by links, somewhat after the fashion of single-trees to double-tree and double-tree to tongue. It was early recognized that this arrangement was unsafe, and the later design, such as shown in Fig. 33, was evidently intended to correct this defect. In the writer's opinion it does not do so. It is not safe, nor does it insure equality of stress.

However improbable the failure of a wire rope may appear to the advocates of its use, it would seem that the possibility of the progressive failure of an entire set of ropes should be eliminated, especially since to do so would lead to a simplification of details and a considerable saving in cost.

The author uses wire ropes for operating lift spans, as shown in Fig. 1. It appears that the working stresses for operating ropes are even greater than those for the suspending cables. An article describing the 273-ft. double-track span over the south branch of the Chicago River† states that the operating ropes were selected in accordance with the following specifications:

"The ratio of the total stress (including bending) to the elastic limit shall not exceed 75%; the ratio of ultimate to direct stress shall not be less than $4\frac{1}{2}$."

It is to be expected that operating ropes so highly stressed will have to be renewed at more or less frequent intervals. Here, again, the high working stresses used seem to have been adopted in order to avoid the difficulties incident to the use of low stresses, such as drums of too large size, or too many ropes.

With such high stresses in the operating cables the stretch of the ropes is con-

* "Mechanical Features of Vertical Lift Bridges", *Transactions*, Am. Soc. M. E., Vol. 40 (1918).

† *Engineering Record*, May 15th, 1915.

siderable. The bridge over the Mississippi River at Keithsburg, Ill., was built with the machinery house at one end of the lift span in order to save metal in the trusses, and, consequently, one set of operating ropes was about 100 ft. longer than the other. The longer ropes stretched much more than the shorter ones, thus causing one end of the span to rise first and producing an uneven, jerky movement. In later designs the machinery house has been placed at the center of the lift span.

There was a failure of the operating ropes on the Hawthorne Avenue Bridge at Portland, Ore., which structure is mentioned by the author on page 589. It appears from press accounts that there were two breaks a few hours apart. After the second break the span moved on up to the top of the towers, being halted only when the counterweights struck the bridge floor. This mishap shows the need for buffers to stop the span in case it gets out of control. The bridge was out of service about 24 hours.

The writer's conclusions with respect to the use of wire ropes may be summarized as follows:

1.—For suspending cables the ratio of the pitch diameter of drum to the diameter of the ropes should be not less than 80.

2.—For suspending ropes the maximum unit tensile stress (including the bending stress) should not exceed one-sixth of the ultimate strength of the finished rope in pounds per square inch.

3.—The suspending ropes should be made of the same length, as nearly as may be, and connected directly to span and counterweight, thus saving the cost of the so-called equalizers, simplifying the details, and perhaps reducing the height of the towers.

4.—Smaller working stresses should be used for operating ropes.

5.—Wherever practicable, other elements should be substituted for wire ropes, even though the cost is somewhat greater, in order to obviate the necessity for such frequent inspection, to reduce the chance of accident, and to lower the cost of maintenance and renewals.

The Pine Bluff Bridge (Table 6) has no wire ropes. Built-up chains are used to support the lift span, which is operated by means of roller sprocket chains. Inasmuch as full descriptions of this structure will be found in the technical press, it is unnecessary to enter into detail here. It provides for a railway track between trusses for Cooper's E-50 loading, and two 11-ft. roadways on cantilever brackets. Fig. 36 is a view of the bridge with the lift span fully raised, and Fig. 37 shows the lift span partly raised and the tower, counterweight and suspending chains. The lift span has a total movement of 49 ft., and is designed to move that distance in 2 min.

The suspending chains are the most interesting feature of this bridge. They consist of links or eye-bars fastened together with pins $3\frac{1}{2}$ in. in diameter and $20\frac{3}{32}$ in. center to center. The main sheaves or drums have a pitch diameter of 8 ft. 7 in., and hence there are eight sets of links in the length of the semi-circumference of the drum. The pins are non-rotatably attached to alternate sets of links, thus confining the rotation to the links having widened heads. The rotatable links are provided with phosphor bronze bushings $\frac{1}{2}$ in. thick. The heads of the links have small bearing surfaces finished to fit the periphery of the drum. The con-

struction of these chains is clearly shown in Fig. 39. These chains are made of medium steel and the working stresses adopted are as follows:

	Pounds per square inch.
Net tension through eye	12 500
Net tension through body, including bending stresses due to bearing on drum	17 000
Bearing on rotating surfaces, bushed with phosphor bronze	6 000
Bearing on non-rotating surfaces	18 000
Bending on pins	20 000

The question as to whether with a bearing stress of 6 000 lb. per sq. in. there will be excessive wear of the links is as yet undecided. Some chains used for ladder dredges have bearing stresses up to 16 000 lb. per sq. in. The rotation takes place through an angle of 22½° in about 4 sec., and the writer believes that the wear will be negligible. It should be noted, however, that it is unnecessary to maintain the pitch as would be the case with sprocket chains, and hence a large amount of wear can occur without causing any trouble. The Pine Bluff Bridge is now seven years old, but the lift span has not been opened very often. It operates in a very satisfactory manner and no repairs have been necessary.

With this type of chain the towers do not need to be so high as for the cable lift bridge and the drums may be much smaller than the wire-rope sheaves.

The weight of the main counterweight is varied by means of the superimposed folding counterweight shown in Fig. 36. This consists of a series of cast-iron disks connected with bolts so as to have a clear distance apart of 1 ft. when the counterweight is extended. This type of auxiliary counterweight is twice as heavy as the chain type attached to the span and the main counterweight, and under present conditions the latter would probable be preferable.

This bridge was let for a lump sum price and no figures are available as to the cost of the suspending chains. The links are made in the same manner as ordinary eye-bars, but there is more machine work for a given weight. It might be thought that such chains are more expensive than wire ropes, but such does not appear to be the case, provided the wire ropes are calculated for conservative working stresses such as those specified by Mr. Leffler. Each of the four chains for the Pine Bluff Bridge weighs about 15 000 lb. and carries a load of approximately 300 000 lb. The four drums and their axles (but not including bearings) weigh about 40 000 lb., and the cast-iron folding counterweights weigh about 84 000 lb. The cost of these three items is to be compared with the cost of the corresponding items for the cable design, it being assumed that no equalizers are used:

Assumed total extreme fiber stress in wire ropes = 32 500 lb. per sq. in. (one-sixth of the ultimate strength).

Assumed pitch diameter of sheave = 12 ft.

Try 1¾-in. ropes; ratio of sheave diameter to rope diameter = 82.3.

$$\text{Bending stress} = \frac{1\,800\,000}{82.3} = 21\,900 \text{ lb. per sq. in.}$$

Direct stress = 32 500 − 21 900 = 10 600 lb. per sq. in.

Allowable direct load per rope = 10 600 × 1.30 = 14 700 lb.

Number of ropes required per corner = 20, say.

Width of drum = 40 in., about.

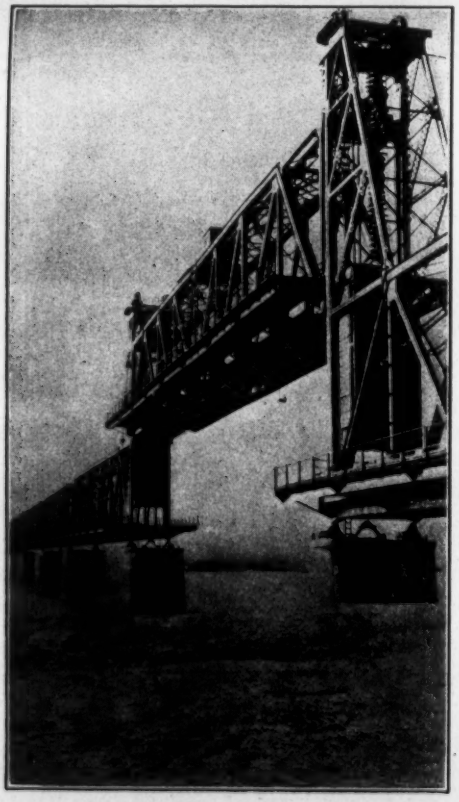


FIG. 36.—PINE BLUFF BRIDGE WITH LIFT SPAN RAISED.

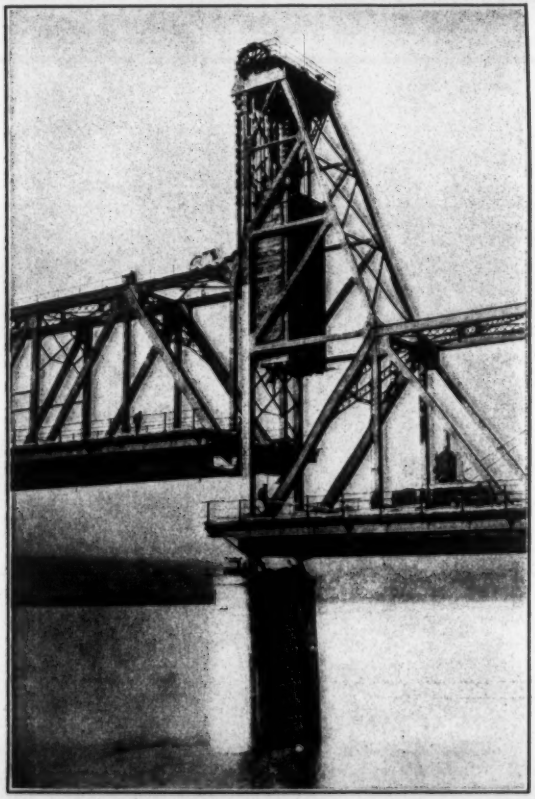


FIG. 37.—PINE BLUFF BRIDGE: WITH LIFT SPAN PARTLY RAISED, TOWER, COUNTERWEIGHT, AND SUSPENDING CHAINS.

The Pine Bluff drum is 24 in. wide and 7 ft. 10§ in. in diameter. The wire-rope sheave will weigh about 2.2 times as much as the chain drum, making the total weight of the wire-rope sheaves = 40 000 × 2.2 = 88 000 lb.

The wire ropes weigh about one-half as much as the chains, hence the corresponding weight of the folding counterweights is assumed to be 42 000 lb. If the chain type of auxiliary counterweight should be used to balance the weight of the suspending chains or ropes, the weight of these chains would be only about one-half as great in each case as the weight of the disk type of counterweight used.

The following are the comparative costs:

Chain Design:	
Chains, 60 000 lb. at $0.20	$12 000
Drums, 40 000 lb. at $0.18	7 200
Folding counterweight, 84 000 lb. at $0.07	5 880
Total	$25 080
Wire Rope Design:	
Ropes and sockets, 48 500 lb. at $0.17	$8 245
Sheaves, 88 000 lb. at $0.18	15 840
Folding counterweight, 42 000 lb. at $0.07	2 940
Total	$27 025

A considerable change in these prices would not alter the conclusion that the chain design is not much, if any, more expensive than the rope design with the working stresses assumed.

In his designs the author uses higher working stresses and employs equalizers. With an extreme fiber stress of 48 500 lb. per sq. in., or about one-fourth of the ultimate strength, there would be required sixteen 1¼-in. ropes at each corner with an 8-ft. sheave or eight 1¾-in. ropes with a 12-ft. sheave. The cost would be about 40% less than in the preceding case if no equalizers are used, and but little, if any, less if equalizers are used. Thus it appears that the relative costs of the chain and cable types depend largely on the working stresses adopted and whether or not equalizers are used.

For very heavy lift spans requiring large sheaves for the wire ropes, the saving in the weight of sheaves effected by using chains is even more marked. The four sheaves for the Louisville and Portland Canal cable lift bridge mentioned in Table 1 weigh 152 tons. The drums for chains would weigh less than one-half as much.

The Pine Bluff lift span is electrically operated, there being a motor at each end. The back-gear shaft is connected with the sprockets through the spur gearing shown in Fig. 39. There are two sprocket chains at each corner of the span, running over idler sprockets and thus forming a loop engaging the drive sprockets. The sprocket chains are attached to equalizer bars and are provided with adjusting bolts, as shown in Fig. 38.

The main counterweights are built up of precast concrete blocks. The construction is clearly indicated in Fig. 37. This photograph shows also the neat appearance gained by giving the tower considerable width in the plane of the trusses and making the tower diagonals parallel to the end post.

Another type of bridge in which no wire ropes are used, is the Strauss direct lift, which makes use of levers for connecting the span and counterweight. Table 6 contains a list of the bridges of this type, which have been built. One of these, the Louisville and Portland Canal Bridge,* is shown in Fig. 40 in the closed position and in Fig. 41 in the open position. The method of counterbalancing is very ingenious. The parallelogram formed by the hanger attached to the U_1 point of the span, the upper counterweight, the lower chord of the counterweight trusses, and the counterweight link which joins the hanger and the counterweight, insures an exact balance at all positions of the span, the counterweight trusses themselves being balanced by the lower or fixed counterweight. The center of gravity coincides at all times with the center of the trunnions at the top of the tower posts.

The span operates by means of four pinions engaging vertical racks on the tower posts. Power is furnished by two 11-h. p. direct-current motors placed underneath the floor near the center of the span and controlled from the operator's house. One minute's time is required to open or close the span.

This type of bridge can be used only in cases where a comparatively small vertical movement is required, such as for inland rivers and canals, since otherwise the counterweight trusses and links would be too heavy and cumbersome. In any case, they are likely to be considerably heavier than the metal in the towers and counterweights of the cable or chain type of lift bridge. On the other hand, there is some saving of truss weight on account of the suspension of the span at the U_1 point, instead of at the end. In many cases the excess cost of structural steel will be largely if not wholly offset by the savings effected through doing away with the wire ropes and equalizers, the counterbalance chains, and the guiding devices, and in some cases there will be a further saving due to erecting the span by the cantilever method without falsework, this procedure being facilitated by the counterweight trusses and links. Broadly speaking, therefore, the question of the type of lift span to use is one of merit and not of cost. It should be said, however, that for special cases like those described by the author in Part III and Part IV, wire ropes must be used.

This type of span requires no guides for the counterweight. The only moving elements of a mechanical nature, the trunnions, are of well-known reliability and may be protected from the weather so as to reduce the maintenance cost to a minimum. This is a decided advantage in the long run.

The author's statement that composite or built-up sheaves are not satisfactory unless the shopwork is better than is ordinarily the case, is not understood. It would seem to be entirely practicable to obtain satisfactory results without taking any extraordinary precautions concerning the quality of workmanship. Mr. Van Cleve describes† certain defects which developed in the sheaves for the Oregon-Washington Railroad and Navigation Company's Bridge (Part IV) and three bridges built in Chicago, Ill., by the Pennsylvania Lines West. In his discussion of this paper (page 1038), Dr. Waddell points out some precautions which were taken to do away with these defects.

The author states on page 582, that where the vertical clearance does not exceed the channel width, the lift span is nearly always cheaper than the swing span or

* This structure was described in *Engineering Record* of August 14th, 1915.

† *Transactions*, Am. Soc. M. E., Vol. 40 (1918), p. 1026.

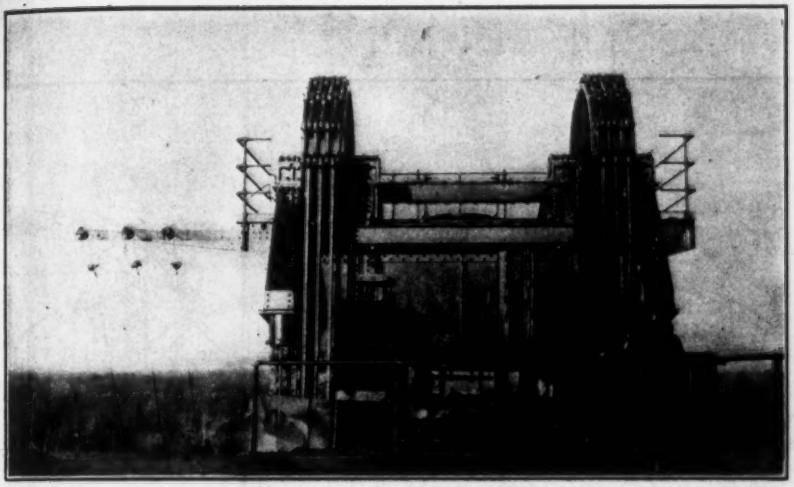


FIG. 38.—PINE BLUFF BRIDGE, SHOWING END OF LIFT SPAN AND TOP OF TOWER WITH SPAN IN FULLY RAISED POSITION.

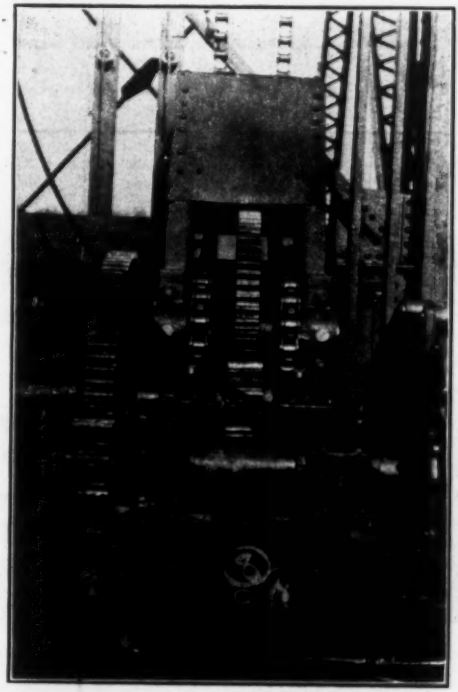


FIG. 39.—PINE BLUFF BRIDGE: REDUCTION GEARS, ETC., AT CORNER OF LIFT SPAN.

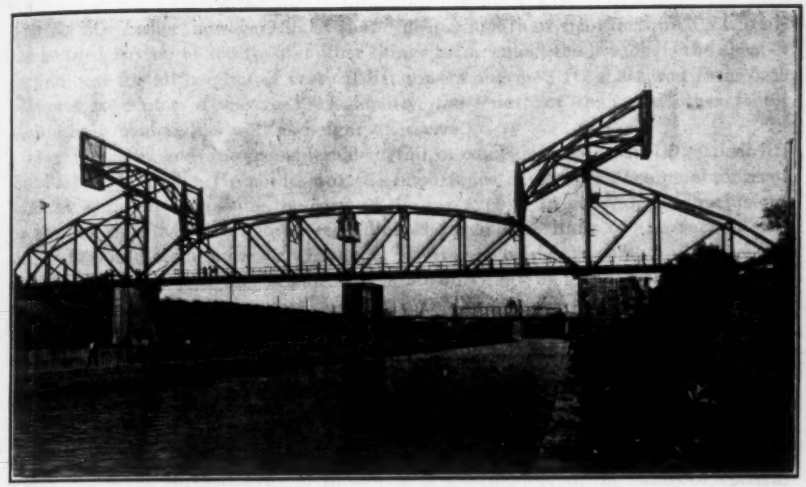


FIG. 40.—LOUISVILLE AND PORTLAND CANAL BRIDGE: LIFT SPAN IN CLOSED POSITION.

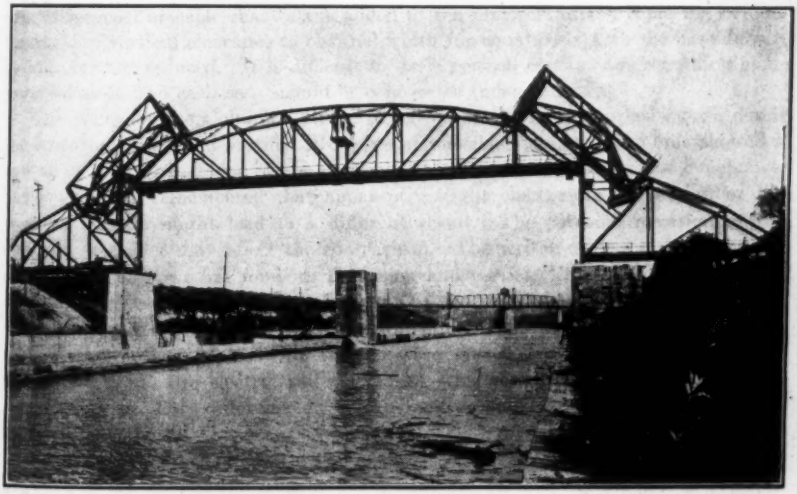


FIG. 41.—LOUISVILLE AND PORTLAND CANAL BRIDGE: LIFT SPAN IN OPEN POSITION.

bascule. This is practically the same as saying that the lift span is more economical than the swing span or bascule if the height of the towers is less than the span length. Mr. Leffler, however, finds* that "for any length of span for which a bascule can be used, say up to 260 ft., and other things being equal, the bascule is the cheaper in first cost for all heights of vertical lift towers above 88 ft., measured from base of tower to center of sheaves." Evidently, the length of span and other factor should be considered as well as height of towers.

Dr. Waddell† goes into considerable detail in comparing the costs of vertical lift and bascule bridges. He points out the importance of the presence or absence of flanking spans in comparing the costs of the two types. In general, his conclusions bear out the author's statement regarding the economic limit of the vertical lift span. It would seem, however, that he is in error in using the same unit prices for both the lift span and the bascule. It is obvious that in many cases the vertical lift span will be more expensive to erect than the bascule, as the former must be erected in a horizontal position, while the latter may be erected in a vertical position, thus saving the cost of falsework. If traffic must be maintained over the channel, thus requiring the lift span to be erected in its elevated position or else clear of the channel on barges and afterward floated into place, the cost of erection may be greatly in excess of the corresponding cost for the bascule. Moreover, the comparative costs are worked out for heavy double-track railway bridges, a procedure which favors the vertical lift. If these points are considered, the ratios of vertical clearance to channel width for equal costs of the two types, will be somewhat less than those given by Dr. Waddell.

It is hardly necessary to point out that in comparing the two types, the cost of maintenance and operation should be considered as well as the first cost. If the capitalized cost of cable renewals is added to the first cost of the cable lift bridge, the ratio of vertical clearance to channel width for equal costs with the bascule will be still further reduced. It is difficult to draw general conclusions regarding comparative costs, and each case should be considered independently.

The plans for the Columbia River Bridge (Part II) were worked out in detail for both the swing and vertical lift types of movable span, and the bids showed a saving of $70 000 in favor of the vertical lift. This would seem to be conclusive, but it must be remembered that apparently slight changes in the design or the working stresses might lead to a different result. The personal equation of the designer or bidder may affect the comparison. The writer made careful estimates of cost for this structure for both the swing and vertical lift designs and found a difference in cost of only $25 000 in favor of the vertical lift, and this difference was largely due to the cost of the draw protection. The writer's design was unlike that of the author in that the trusses were placed only far enough apart to accommodate the two electric railway tracks, with slow-traffic roadways on cantilever brackets. This difference in design may account for part of the difference in comparative costs.

The statements made by the author regarding the cost of operation of the vertical lift span of the Hawthorne Avenue Bridge require further explanation in order to be convincing. Apparently, the cost of power is the only item taken into account.

* *Bulletin 165*, Am. Ry. Eng. Assoc., March, 1914.

† "Economics of Bridgework," p. 297.

If the cost of maintenance and repairs should be considered, the comparison might be different. The entire set of operating cables was replaced in 1914 on account of two breaks, as heretofore mentioned, and the item of cable renewals alone may increase the operating expenses considerably. Nothing is said in regard to the comparative weights and lengths of the various spans, the comparative frequency of operation, or other factors which might be of importance in forming a conclusion as to the relative economy of operation of the different types compared.

The author attributes the low cost per operation to the stated conditions that the lift span need be lifted only high enough to clear the approaching boat (often only 15 or 20 ft.), and that there are no wedges or locks to be driven after the span is seated. In so far as these points are concerned there does not appear to be any particular difference between the vertical lift and the bascule. It is a fact that bascule bridges do not open fully for small boats. Operating records show that single-leaf bascule spans having clearances as small as 15 ft. are opened only about 27° for tugs and canal barges. The angular movement of double-leaf bascules will be even less, since the maximum clearance is at the middle of the channel. The Polk Street Bridge over the south branch of the Chicago River, a double-leaf bascule, operates 3 100 times per year with an average angle of opening of 35 degrees.

The writer is under the impression that the Broadway Bridge bascule span referred to by the author has considerably more vertical clearance when closed than either the swing or the lift span. If so, it may be that the bascule span opens infrequently or not at all for small boats, thus making the cost per operation comparatively large, both on account of the less frequent operation and the greater amount of movement required when opened.

The vertical lift span does not possess any advantage over the single-leaf bascule span (save a few types of the latter) in respect to the requirements for end locks. The operating mechanism of the bascule is fully as capable of holding the span down as the operating cables of the lift span. The end locks serve to hold the span firmly and in correct position on the piers, and they are just as desirable for the lift span as for the bascule, and must be used at both ends of the former, though required at only one end of the latter.

In order to obtain a reliable basis of comparison of operating costs, the bridges compared should be as nearly alike as may be in respect to traffic capacity, clear channel width, vertical clearance when closed, and general dimensions. There are two cases of bridges in Chicago in which these requirements seem to be met. The 210-ft. Lake Shore double-track cable lift bridge over the Calumet River is adjacent to the 235-ft. Baltimore and Ohio double-track bascule bridge. The former span has two 125-h. p. alternating-current motors and the latter, although 25 ft. longer, has two similar 100-h. p. motors. The cable lift has chains to counterbalance the cables, so that no excess power is required on account of unbalanced cable load. Over the south branch of the Chicago River there is a lift span 273 ft. long built by the Pennsylvania Lines, and a short distance away there is a bascule span 260 ft. long. Both bridges carry double-track loading and operate in practically the same length of time. The lift bridge was designed to operate with unbalanced cables and is provided with two 300-h. p. direct-current motors. Later, it was found desirable to install counterbalance chains. The bascule span has two 150-h. p. direct-current motors. The Louisville and Portland Canal cable lift bridge is of

about the same length and weight as the last mentioned cable lift bridge. It operates with two 150-h. p. alternating-current motors, but its movement is only about one-third as great as that of the Chicago bridge in the same length of time, 45 sec. It would be interesting to compare the operating costs of the bridges mentioned in the foregoing cases.

Much of the designing of movable bridges is in the hands of patentees, who naturally enough are quick to call attention to the advantages of their respective types and slow to point out the deficiencies thereof. Much might be said respecting the comparative merits of the different kinds of movable bridges, but inasmuch as the governing conditions vary so much for different bridges it is advisable to consider each case as a separate problem, choosing the most satisfactory type and design of movable span in the light of all the conditions and circumstances pertaining to that particular structure. If an æsthetic appearance is of primary importance, certain kinds of movable spans are ruled out at once. A type that would be as satisfactory as any under present conditions might be rejected in favor of some other type which could be more easily adapted to future changes.

The vertical lift bridge unquestionably has a wide field of application, and it is hoped that the author's paper and the discussion thereon will result in a fuller knowledge of the advantages and disadvantages of bridges of this type and a clearer understanding of the more important requirements and considerations governing their design and construction.

HENRY C. TAMMEN,* M. AM. SOC. C. E. (by letter).—With all the progress in the design and construction of bridges made within the last twenty-five years, it is the writer's opinion that one of the important steps has been the use of reinforced concrete as one of the principal construction materials for superstructures. The use of concrete as the primary construction material for superstructures of bridges is still increasing; and although its application as the primary material is not always possible or practicable, its use for floor construction on large highway bridges is almost universal. The greater permanence and absolute fire-proof qualities of such construction makes its use highly advisable, and to this the large fire losses on bridges with inflammable wood floors, which have been experienced in recent years, bear testimony. It is somewhat surprising, therefore, to note the little regard usually given to the possibility of the use of a fire-proof concrete floor on the movable span for a bridge when for the remainder of the structure nothing else would be countenanced. The ordinary procedure appears to be "to put the cart before the horse"; that is, to select the type of movable span first and then to be content with the possibilities of that type as regards floor construction, rather than to reverse the process and select a type of movable span favorable to the same permanent fire-proof floor which is required for the remainder of the structure. This criticism applies principally to highway bridges, for only a few railroad companies consider it economically advantageous to use other than open-deck timber floors on any part of their larger bridges.

The possibility offered by the vertical lift bridge for the use of such a permanent fire-proof concrete floor, deserves considerable emphasis, and it is the writer's opinion that future types of movable spans must be adapted to such construction.

The writer has been identified with the design of movable spans of various

* Kansas City, Mo.

types for a great many years and particularly with the vertical lift span more than with some of the other types. He has had his troubles in estimating quantities and costs of movable spans of various types where time did not permit detailed approximate designs to be made; other bridge engineers no doubt have had the same troubles to contend with, for engineering literature as regards cost of movable spans is exceedingly scant, and it has been necessary for each man to develop his own data as necessity and opportunity offered. The writer believes, therefore, that cost data of vertical lift bridges will be of considerable benefit to the Profession

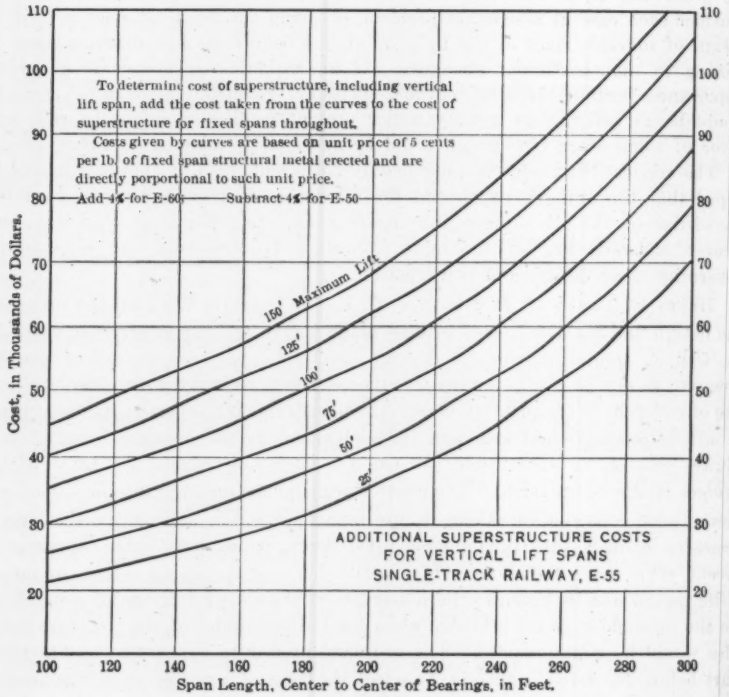


FIG. 42.

and offers herewith data covering the costs of single and double-track railway vertical lift spans.

For the superstructure, the curves on Figs. 42 and 43 give directly the costs in dollars required to convert an ordinary fixed span into a vertical lift span. These costs include towers, counterweights, machinery, ropes, concrete machinery house, electrical operating equipment, signal lights, and all other superstructure parts, including metal that must be added to the fixed span in order to change it to a lift span.

It is manifestly impossible to give similar data for substructures on account of the variety of conditions which might be encountered. As being of most service, the writer has given for the substructure the total additional load, in tons, that must be carried on the lift span piers. (Figs. 44 and 45.) With this information, the necessary changes in piers to provide for the lift span construction can readily be made. For instance, suppose a 200-ft. double-track vertical lift span of 100 ft. vertical lift and Class E-55 loading is under consideration. The curves (Figs. 44 and 45), show an added load on piers of 1 106 tons, or 503 tons for each

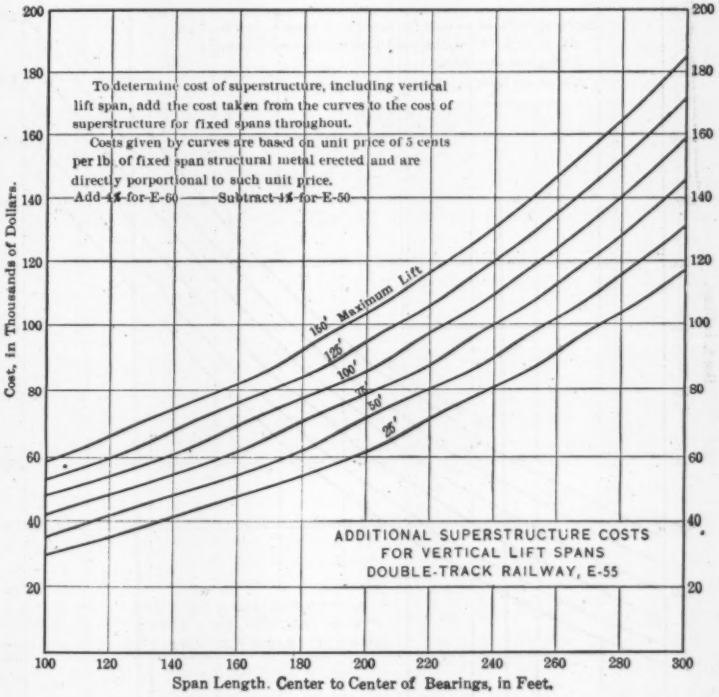


FIG. 43.

pier, or 503 000 lb. on each shoe. The pier shoes of the lift span will be proportioned only for live load and impact, the dead load being transferred to the tower shoes. Each tower shoe will be proportioned for ordinary dead and live load on the span supporting the tower plus the dead load previously given (503 000 lb.), plus the dead load of an ordinary fixed span of the same length as the lift span (in this case, one-fourth of 1 156 000 lb. = 289 000 lb.). Having proportioned the shoes on the piers, the size of the pier tops can readily be determined and the remainder of the piers designed; ordinarily, the sizes of the pier tops are increased very little by the conversion of one of the fixed spans into a lift span.

An explanation of the assumptions made in developing the superstructure curves (Figs. 42 and 43) is desirable. A study of the actual costs of many lift bridges showed that the unit costs of the various items making up the superstructure cost, such as machinery, ropes, counterweights, metal in towers, etc., had fairly constant ratios to the cost of the ordinary fixed span metal in the same

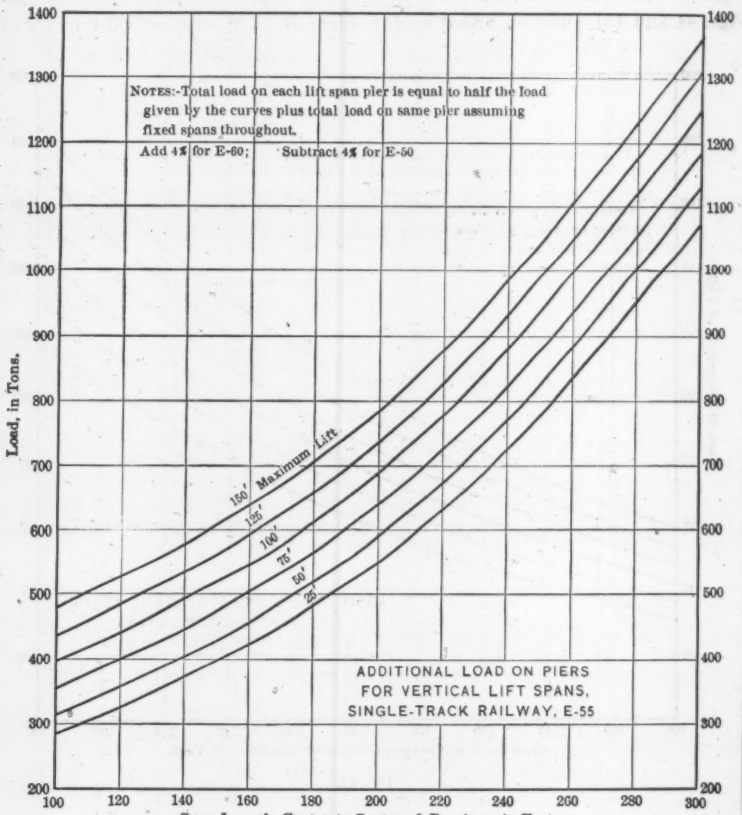


FIG. 44.

structure. These constant ratios were first determined and then the costs calculated on the basis of 5 cents per lb. for ordinary fixed span metal erected in place. If, for instance, for any particular bridge, the fixed spans erected are estimated to cost 6 cents per lb., the costs given in the curves (Figs. 42 and 43) should be multiplied by the ratio, $\frac{6}{5}$, or increased 20 per cent.

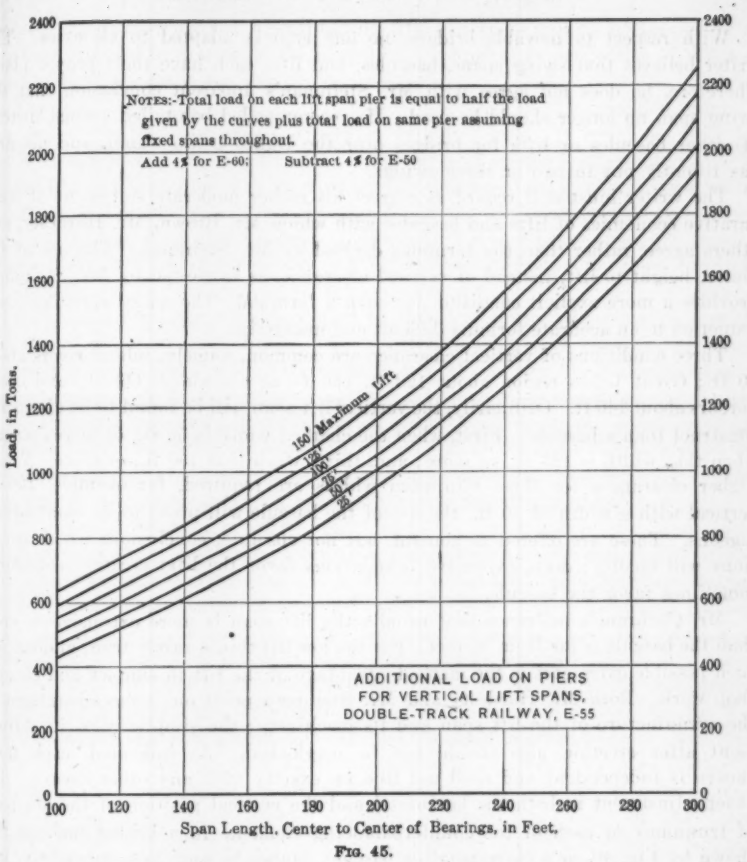


FIG. 45.

ERNEST E. HOWARD,* M. AM. SOC. C. E. (by letter).—The object of the paper was not to be controversial as to types of movable bridges, but to compile a reasonable record of an important bridge development for the Profession. The features and details brought forth in the discussions add in interesting ways to that record. In explanation of the omissions which are questioned or criticized, the writer must plead attempted brevity and condensation which necessitated dismissal "with a sentence or two", of more than one important matter.

Details other than those of movable spans are included in Parts II, III, and IV, for in these sections exceptional structures with many features worthy of record are described. The title of the paper might more correctly have read, "Vertical Lift Spans and Other Noteworthy Features of Some Bridges in Which They Are Used", as indicated by the first sentence of the synopsis. Two of these bridges are examples of unusual and highly specialized achievements, apparently perfect solutions of the problems presented.

* Kansas City, Mo.

With respect to movable bridges, no one type is adapted to all cases. The writer believes that swing spans, bascules, and lifts each have their proper place. Therefore, he does not agree with Mr. Steinman's apparent conclusion that the swing span no longer should be used. He recommended bob-tailed swings instead of either bascules or lifts for bridges over the Dalles-Celilo Canal, and his firm has recently put in two or three swings.

The writer must still regard as correct his rather moderate statement of comparative economies of lifts and bascules with which Mr. Brown, Mr. Hardesty, and others agree, rather than the formulas derived by Mr. Steinman. The use of the factor, height to lift, instead of vertical clearance, as suggested by Mr. Steinman, provides a more explicit condition for such a formula. The many variables make reduction to an accurate formula difficult and uncertain.

Three conditions of vertical clearance are common, namely, inland rivers about 50 ft.; Great Lakes region about 120 ft. (90 ft. at Cleveland, Ohio); and ocean harbors about 140 ft. Ordinarily, the vertical lift span will be found to be cheaper to construct than a bascule: First, when the channel width is 50 ft., or more; second, when that width is 125 ft., or more; and, third, when it is 150 ft., or more. Where higher clearances for these minimum widths are required, for example, 100 ft. vertical with a width of 50 ft., the use of the bascule will prove to be more advantageous. These are offered as general, but not absolute, conclusions which conditions will modify; deep, expensive foundations favor the lift; erection conditions sometimes favor the bascule.

Mr. Cochrane's inference that usually the lift span is more expensive to erect than the bascule is far from correct. For the low lift this is rarely true; and against such possible extra cost is the marked advantage of the lift in simpler and cheaper shop work. Both Mr. Hardesty and Mr. Steinman point out some advantages in the manufacture of the lift span and its machinery; the simplicity of its adjustment after erection also should not be overlooked. As indicated, each main sheave is independent and need not line up exactly with any other sheave. The exact adjustment in both the horizontal and the vertical position of the six pairs of trunnions in each of the counterbalancing elements in a bridge such as that shown by Fig. 40, or a corresponding bascule, cannot be easy, yet, apparently, it is essential to smooth operation. Mr. Hardesty advises that for the Portsmouth Bridge, where erection conditions greatly favor the bascule and bids on both a lift and a double-leaf bascule were received, the low bid on the superstructure alone, erected in place, was $66 000 less for the lift.

Mr. Cochrane refers to certain general comparisons of lift and bascule bridges made by B. R. Leffler, M. Am. Soc. C. E., in 1914. The writer understands that these comparisons considered only two or three lift bridges, one of an abandoned design, and all with towers of four sheaves. The conclusions, therefore, cannot properly apply to the usual arrangement of towers.

Mr. Tammen's discussion is a most valuable addition to the paper, particularly his cost curves (Figs. 42 and 43). These have been compared and check closely with the actual costs of many of the structures already built. They give results certainly close enough for preliminary estimates. It is necessary only to estimate the cost of a bridge with fixed spans throughout, and then to add the costs for the movable features from the curve and, if necessary, increase the piers, for the added

loads. This simple method of estimating lift-span costs will be of service to an engineer inclined to determine for himself the comparative economies of different types of movable spans for any given conditions.

The very ingenious operating equipment described by Mr. Hayden seems to be admirably adapted to small structures with low lifts. It could be used, but not so satisfactorily, for higher lifts. The far end of the span would move by oscillations and not smoothly, because of the difference in length as well as the difference in the stretch of Ropes *A* and *B*. Since Mr. Hayden's discussion was received, his operating plan has been analyzed for a rather heavy span, 120 ft. long, with a 40-ft. lift, now under design, but the unequal stretch of the ropes precludes its adoption.

This is the condition that developed at the Keithsburg Bridge which was the first large low-lift structure built. The operating machinery is over one end panel so that the up-haul and down-haul ropes to the far end are 230 ft. longer than to the near end. The extra flexibility of this 230 ft. of rope permitted the far end to oscillate up and down slightly during operation. A simple solution of the difficulty was to put idler sheaves near the middle of the span and to replace the operating ropes of the near end with longer ones doubling over the idler sheaves, thus making the rope conditions for the two ends practically alike.

Mr. Green's observations carry weight from the standpoint of the owner interested not only in the construction but in the maintenance and operation of bridges and other mechanical structures. It might be noted that these openings through which "boats operate with facility in swift current" are unhampered by long fender constructions, which have never been found necessary with lift spans, even where vessel movement is frequent and the tows are as long as 2 000 ft.

That Mr. Walker's conclusions are justified in reference to the comparative immunity of the lift span from serious damage by boats, is proven by Mr. Van Cleve's description and illustration of the damaged bottom chord. For some years the writer has been seeking the opportunity suggested by Mr. Walker of developing the architectural treatment of a lift span, particularly in order to show what can be done with reinforced concrete towers. The towers of the Tower Bridge in Mr. Walker's own city (London, England) might more rightly support a lift span than stand as guardians of the small bascule leaves.

Mr. Walker suggests smallness of sheaves as contributing to failures of ropes operating swing bridges, but he does not disclose all the conditions. It may readily be that such failures are in large part due to the deformation of the rope as it runs upon, and as it leaves, the flat surface of an ungrooved drum. Ropes under considerable tension and not running in grooves are subject to rapid internal wear, and the importance of the proper design of grooves and their relation to rope life should be emphasized. The grooves should be just sufficiently larger than the rope to prevent seizure, but they should afford all possible support to prevent the flattening out deformation.

Mr. Van Cleve prefers the separate centering castings which are admittedly desirable for very wide spans with tracks in the center. The flaring tower guides, however, can serve the same purpose more cheaply if provided with adjustment. Motor-driven rail joints as described and illustrated by Mr. Van Cleve can be used, but the simpler immovable joints have proven entirely satisfactory. Plate I

shows a simple rail joint combined with interlocking devices operated from the movable span.

The writer is pleased to see the listing of additional lift bridges (with some duplication) by Mr. Hardesty and Mr. Cochrane. Full information concerning such bridges was not in the writer's hands, which is also true of two or three other bridges not included in any of these lists; however, substantially all the important bridges are now included.

Mr. Hunter's experiences on the Don River Bridge in Russia should be both interesting and stimulating to the young engineer. Wholly alone in his responsibility, he met and solved its problems with the materials, equipment, and labor at hand. The ingenuity of the home-made cranes for lifting the 20-ton sheaves will be commended by many engineers who have confronted similar difficulties far from supplies of suitable equipment. His recital of the rope replacement experiences bears out the statement that ropes can be replaced with comparative ease.

Some of the questions raised by Mr. Cochrane have already been answered by Mr. Steinman, Mr. Van Cleve, Mr. Hardesty, and others; but the careful analysis which Mr. Cochrane has made of some of the fundamentals of lift-bridge design deserves a complete and studied reply. Mr. Cochrane's surmise concerning the writer's use of the word "may", in reference to the equalizers, is correct. The word was used intentionally, for although most—but not all—of the bridges listed have equalizers, they could in large part have been omitted. The omission of equalizers was first proposed to the writer about ten years ago by Mr. C. F. Swigert, and an analysis similar to that given by Mr. Cochrane justified the proposal. Experience verifies the conclusion, for an examination of many equalizers after service has disclosed few, if any, which were apparently "out of level", except in some cases for the first bars. Future design tends toward connections which may be adjusted at intervals by special jacks. The equalizer shown (Fig. 33), offers no facility for the removal of ropes, that could not be duplicated by direct connections. However, this equalizer will distribute the load with approximate uniformity to all the ropes; of course, with triangular shaped bars, exact balance does not obtain. The specification which Mr. Cochrane quotes seems to be principally a description of equalizers which have been used. More than equalizer design is needed to provide for rope replacement.

The experiences with sheaves referred to by Mr. Van Cleve were just the reason for the writer's statement that composite sheaves not fully riveted were not satisfactory; for they were not. The bearing fits provided by the shops were so poor that the repairs described by Mr. Van Cleve were necessary. Such sheaves can be made, but fully riveted composite sheaves will come from the ordinary shop in better shape.

Exception must be taken, however, to Mr. Cochrane's conclusions concerning proper rope stresses and sheave diameters. He falls into the academic error of considering theories instead of experiences. He does not cite any failures of ropes to prove incorrect the writer's statement that "sheaves of 60 to 80 diameters afford satisfactory proportions, etc.", nor to show that the working stresses on ropes used on the bridges built are not fully conservative. His position seems to be that although the long list of bridges may be in satisfactory service, they should not be, because they do not meet his theoretical deductions. The answer

PLATE I.
TRANS. AM. SOC. CIV. ENGRS.
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HOWARD ON
VERTICAL LIFT BRIDGES.

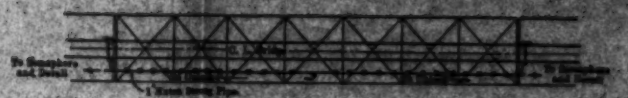
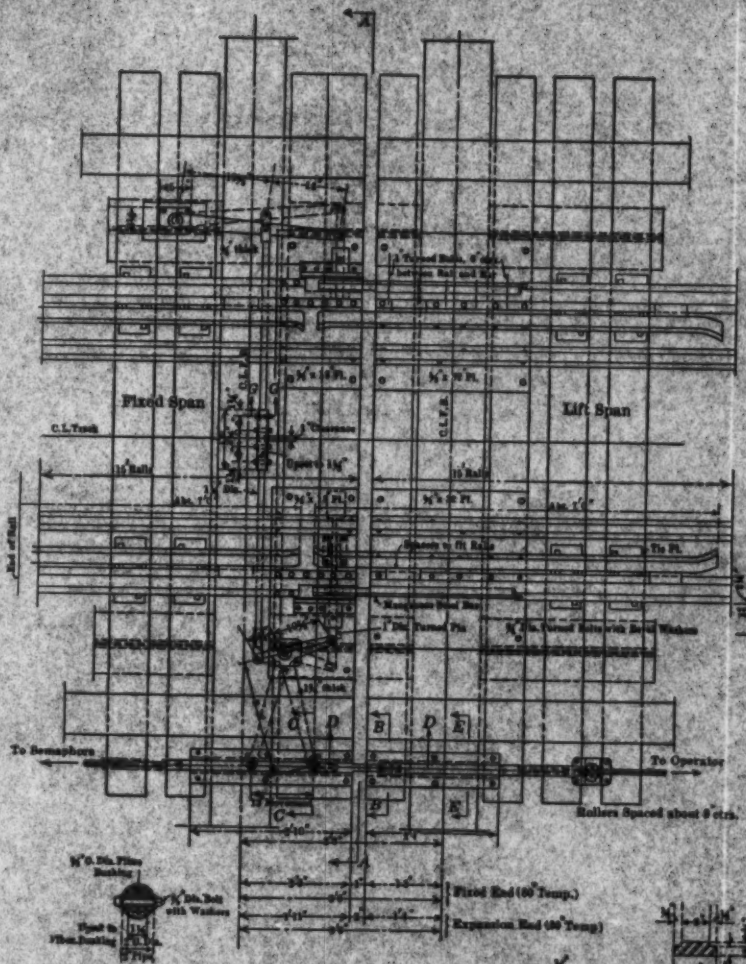
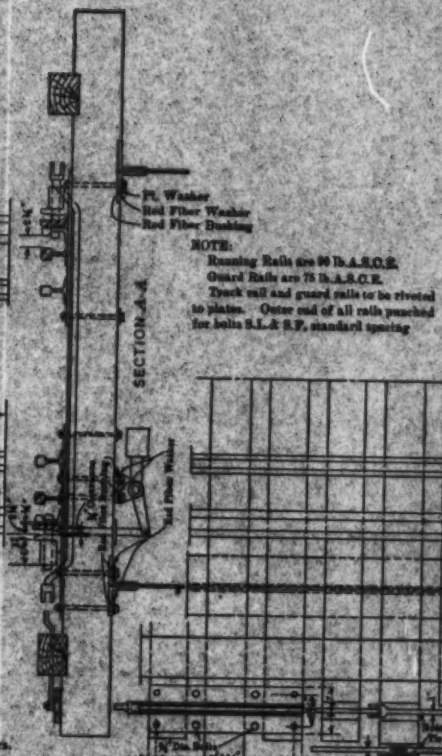


DIAGRAM FOR LOCATION OF RAIL LOCKS

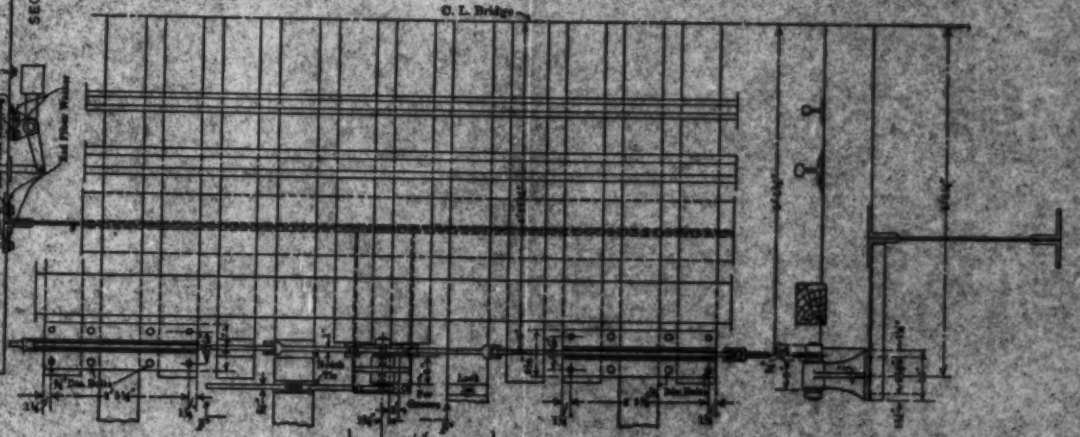
LIFT BRIDGES

RAIL JOINTS AND INTERLOCKING DEVICES



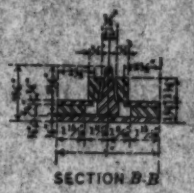


NOTE:
Running Rails are 90 lb. A.S.C.E.
Guard Rails are 75 lb. A.S.C.E.
Track rail and guard rails to be riveted to plates. Outer end of all rails punched for bolts S.L. & S.F. standard spacing

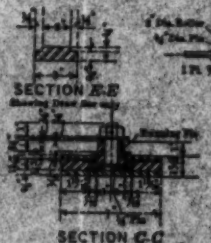


SECTION G-G

SECTION D-D

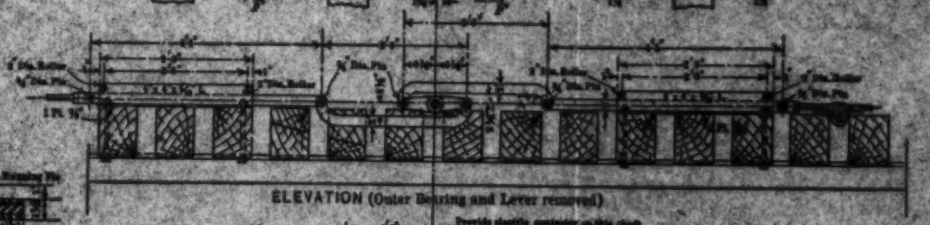


SECTION B-B



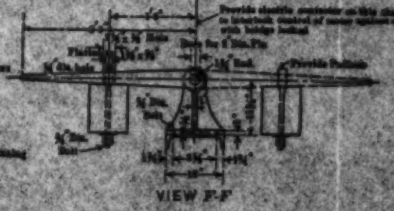
SECTION E-E

SECTION C-C



ELEVATION (Outer Bearing and Lever removed)

DETAIL OF ROLLER AND BASE



VIEW F-F

is that they are. It would seem to be more reasonable to deduce a practice from what has been accomplished.

Mr. Cochrane evidently recognizes the absurdity of calculations which would require two hundred 2¼-in. ropes for the Harriman Bridge instead of the sixty-four 2¼-in. ropes used (which after about ten years of service show no signs of distress). Taking the breaking strength of a 2¼-in. rope as 190 tons (the Roebling catalogue gives 210 tons), the ultimate strength of the 200 ropes would be 38 000 tons—and this to support 1 700 tons; or, with a load factor of 5, one 2¼-in. rope would support safely 38 tons; 45 ropes would support 1 700 tons. In effect, then, the other 155 ropes proposed would function only to take bending stresses!

Mr. Leffler's specifications for sheave sizes and for ropes were prepared some years ago when there were comparatively few lift bridges in service and when his own experience with them was limited. The writer believes that Mr. Leffler would now consider those specifications ultra-conservative.

The wide variation (nearly 250%) of the bending stress formulas cited by Mr. Cochrane and others, indicates the difficulty of reducing complex conditions to a formula. It can safely be assumed that bending stresses of the wires in a rope do not exceed those in a single untwisted wire. Of course Reuleaux, Rankine, and Unwin are incorrect, and are merely on the safe side, in assuming such stresses to be equal. If a closely coiled spring is bent, the distortion occurs mainly by torsion of the wire and has little similarity to bending stress. As the spiral becomes flatter, the stresses begin to approach simple bending, but they remain a combination of bending and torsion. Also, a wire in position of extreme fiber tension on one side of a rope is, within a short distance, on the compression side; and the relative movements of different elements of a rope may affect the wire stresses. The other formulas attempt to meet these and other peculiarities, and severally conclude, for 6-19 rope, the real stresses to be 4%, 6%, 20%, 56%, and 58% less than the bending stresses of a single untwisted wire. Again, the best guide seems to be experience guarded by a knowledge of maximum stresses which certainly cannot be exceeded.

Unfortunately, Mr. Brown in his interesting but brief discussion did not discuss rope stresses, although he probably has been responsible for more permanent operating machines using wire rope than any other engineer. The writer's recollection of Mr. Brown's practice, from a conversation some years ago, is that for sheaves of 60 diameters or more he ignores the bending stresses and selects ropes with an ultimate strength of six or eight times the total direct load. This might be three or four times the sum of the bending and direct stresses, as calculated by the most conservative of the formulas. Mr. Brown remarked that every one knows a short length of wire rope will lie limply over a sheave, or will lie on the floor in a loop of less than 60 diameters without straightening out, because of the extreme fiber stress.

The writer hesitates to dictate specifications for design, for he does not believe that exactly the same working stresses should always apply. A bridge which is likely to be operated 100 times a day differs from one which will be operated 6 or 7 times a year. The following specifications, in abbreviated form, as used by the firm of Harrington, Howard, and Ash in many designs, must be considered in

connection with corresponding specifications of materials and workmanship which are also given herewith in abbreviated form:

Designing Specifications.—
Suspending ropes shall be not less than $\frac{7}{8}$ in. in diameter for 6 by 19 rope, nor $1\frac{1}{2}$ in. in diameter for 6 by 37 rope. Operating ropes shall be not less than $\frac{5}{8}$ in. in diameter for 6 by 19 rope, nor $\frac{7}{8}$ in. in diameter for 6 by 37 rope.
Ratios of sheave diameter to rope diameter shall be not less than:

	Suspending ropes.	Operating ropes
6 by 19 rope	60 to 80	30 to 48
6 by 37 rope	48 to 60	24 to 36

Ratios of ultimate strength of rope to direct stress (not including bending) shall be not less than 7 to 9 for suspending ropes; and 4.5 to 6 for operating ropes.
Ratios of ultimate strength of rope to direct stress plus calculated bending stress shall be not less than 3.5 to 4.5 for suspending ropes; and 2.25 to 3 for operating ropes. When bending stresses are calculated according to the following formulas:

a = rope area, in square inches;
d = rope diameter, in inches;
D = sheave diameter, in inches;
B = bending stress.

For 6 by 37 rope:

$$B = \frac{1\,700\,000\, a \times d}{D}$$

For 6 by 37 rope:

$$B = \frac{1\,215\,000\, a \times d}{D}$$

In general, the lower values may be used for bridges which are seldom operated, and the higher values for those which are frequently operated, or as other conditions may warrant.

"Materials and Workmanship (Abbreviated).—
"Wire ropes shall be of special plow steel wire. Unless otherwise provided on the drawings, suspending ropes shall have 6 strands each of 19 wires, laid around a hemp center; operating ropes shall have 6 strands each of 37 wires, laid around a hemp center. The minimum tensile strength of wires per square inch shall be: For wires, 0.10 in. or less in diameter, 235 000 lb.; for wires from 0.10 in. to 0.125 in. in diameter, 230 000 lb.; for wires more than 0.125 in. in diameter, 225 000 lb."

The specifications provide further requirements for wires, elongation, twisting, repeated bendings, etc., and for rope manufacture. Provisions are made for tensile tests to destruction of samples of the finished ropes, and the number and methods of making tests are fixed. The minimum ultimate breaking strengths of wire ropes, in pounds, are required to be as follows:

Diameter of rope, in inches.	6 by 19 rope.	6 by 37 rope.
$\frac{5}{8}$	33 000	.....
$\frac{3}{4}$	45 000	.....
$\frac{7}{8}$	61 000	52 000
1	80 000	68 000
$1\frac{1}{8}$	101 000	86 000

Diameter of rope, in inches.	6 by 19 rope.	6 by 37 rope.
1¼	121 000	103 000
1⅜	147 000	125 000
1½	173 000	147 000
1⅝	200 000	170 000
1¾	230 000	195 000
1⅞	264 000	224 000
2	297 000	262 000
2⅛	335 000	285 000
2¼	375 000	319 000
2⅜	417 000	355 000
2½	470 000	400 000
2¾	560 000	476 000

Specifications for sockets, methods of attaching them, and other details, are included, and lengths are limited as follows:

"The length of each rope shall be marked on the socket; this measurement shall be made while the rope is supported throughout its length in a straight line and under a tension of 4% of the specified minimum ultimate strength; a maximum variation of ¼ in. per 100 ft. will be allowed."

It should be noted that these required rope strengths are less than the values given in manufacturers' catalogues. On the basis of catalogue values, the ratios of ultimate to direct and total stresses will be better than those provided by the designing specifications. In any event, the results with these specifications, both for materials and for design, have been satisfactory.

The writer must refute Mr. Cochrane's implication that the many bridges built were not conservatively designed, because the rope stresses exceed a factor which he arbitrarily establishes. His conclusions in the light of actual experience should read somewhat as follows:

1.—Experience shows very satisfactory results and no failures of suspending ropes on sheaves of 60 to 70 rope diameters.

2.—Experience shows satisfactory results in the use of ropes with an ultimate strength of three and one-half to five times the direct stresses plus the bending stresses, according to extremely conservative formulas.

3.—Operating ropes with the higher stresses and over smaller sheaves are satisfactory.

4.—No other elements can satisfactorily and economically replace wire ropes, first cost, maintenance, and replacement considered.

Certainly, the ropes must be lubricated, and internal lubrication must be possible, otherwise the ropes in the Halsted Street Bridge could not have lasted 27 years without it, no matter how large the sheave proportion.

Mr. Cochrane's concern for the successive failures of a group of ropes by reason of shock or jar thrown on the second by the failure of the first, will disappear when he considers that a failure of wire rope under tension usually occurs with a comparatively slow stretching and letting down.

The accident at the Hawthorne Bridge to which Mr. Cochrane alludes was due to the thoughtless removal of the paving blocks from the lift span without

lightening the counterweights or adding an equivalent weight to the span. When operation was attempted, the unbalanced span got beyond control and was carried to the tops of the towers, stopping there with a mighty bounce when the counterweights struck the floor, which shock broke some of the operating ropes. It is the writer's recollection that only two of the eight operating ropes were broken. Thus, there were two hoisting ropes about 1 in. in diameter, 240 ft. long, to be purchased, and they were as easy to place as to reeve on an ordinary derrick. Surely, no one would think of that expenditure as an extraordinary or extravagantly expensive maintenance cost after an avoidable accident, from which the main suspending ropes and other details showed no damage. In the writer's opinion buffers would not have affected the conditions at all. The moral of that accident is not that smaller working stresses should be used for operating ropes, but that movable spans are big expensive machines which should be cared for competently.

The Pine Bluff Bridge is interesting as an example of what can be done to build a lift bridge without using wire rope. It seems to be the only one of its kind.

It is difficult to understand how Mr. Cochrane, so conservative with rope stresses, is willing to be so generous with chain stresses. The links of the chains described in his discussion are of medium steel presumably of an ultimate strength of from 55 000 to 65 000 lb. per sq. in., or from 3.24 to 3.82 times his working stress of 17 000 lb. per sq. in., instead of the six times which he considers necessary in ropes. Conservative designers use 1 500 lb. per sq. in. in the bearing pressure of trunnions or pins in bronze bearings, and 3 000 lb. per sq. in. is rarely exceeded, yet Mr. Cochrane uses 6 000 lb. per sq. in. Mr. Leffler's specifications allow 1 500 lb. per sq. in. for axle steel on phosphor-bronze bearings, on which the speed is slow and intermittent, or, in ordinary cases, with parts moving at moderate speeds, for tool steel on bronze 900 lb. per sq. in., and for structural steel on bronze, 600 lb. per sq. in. If the chain pins described by Mr. Cochrane are of structural steel, apparently, he would need ten times his present width of chain to meet this specification, or if they are of axle steel four times his present width. Surely those links must screech as they turn, unless each separate pin is oiled before each operation. Would not the adequate lubrication of such a chain under such heavy stresses be much more difficult than the dressing of wire ropes?

In his comparison of chains with ropes, Mr. Cochrane selects twenty 1¾-in. ropes per corner for a span weighing 600 tons. The Hawthorne Bridge has twelve 1½-in. ropes per corner and weighs 850 tons. It has been in service for 10 years, has been operated more than 50 000 times, and the ropes are in good condition. A report on the chains after 50 000 operations would be interesting. If Mr. Cochrane will compare chains and ropes designed with the same factors of safety, with the same relative conservatism of machinery design, he will find that, instead of being equal, the rope design is far cheaper and simpler than chains.

The fallacy of rough comparisons based on meager data is evident in Mr. Cochrane's recital of sizes of motors on different bascules and lifts as relating to power requirements. It happens that for all the lift spans mentioned, motors in excess of power requirements, in some cases twice as much, were used,

in order to have sufficient excess of power and capacity in one motor to operate if the other was burned out or removed for repair. Mr. Van Cleve states that less power is needed for the operation of a lift than of a bascule, which Mr. Cochrane can assuredly verify from a theoretical standpoint, with much greater accuracy than by comparing motors which happen to be on different spans. The counterbalance chains referred to were used primarily to keep down the peak draft on the storage batteries utilized as sources of power.

The writer has been informed that the general practice in Chicago is to require full opening of bascules; this does not agree with Mr. Cochrane's data of the Polk Street Bridge in that city. The Pilots Association in Portland, Ore., objected to the partial opening of the double-leaf bascule span of the Broadway Bridge (250 ft. wide), and secured full opening, because they had no way of determining either the vertical or the horizontal clearance—or the horizontal clearance at any given height above the water—of the partly opened leaves. They never objected to the partial opening of the lift spans because the horizontal clearance is always the same, and the vertical clearance can be seen on the scales on the towers.

Table 7 contains additional data relative to the four Portland bridges mentioned in the latter part of Part I of the paper, which will answer in some measure Mr. Lindenthal's request. These bridges all serve the same kind of traffic, are well maintained and operated, each is excellent of its kind, and there is some rivalry between the operators of the different spans for speed. The structures, therefore, are more comparable than bridges in separate localities and operated under different conditions. The measurement of the current consumed was made by the Portland Railway, Light and Power Company which supplies the power. The writer has timed bridges in many places, but he has never known the time which intervenes between stopping traffic to starting it again after boat passage as brief as it is for the Hawthorne Bridge, although the span is 245 ft. long.

On several occasions he has seen a steamboat pass the Morrison Bridge, travel the 1 200 ft. to the Hawthorne Bridge, and the latter would open, pass the boat, close, and have traffic moving before the Morrison Bridge had traffic moving. The Morrison swing is an excellent swing span operated swiftly but, of necessity, opening full for every boat, even for those for which the Hawthorne Bridge would lift only 12 or 15 ft.

The costs depending on number of operations is low for all the bridges. The fixed or overhead cost inherent in a movable span varies with locality, working conditions, salaries paid, etc. Public employees in Portland cannot exceed 8 hours per day, or 48 hours per week, so three shifts are employed, plus a relief shift. One operator living near a bridge can sometimes do all the operating—night and day—and attend to the maintenance of the machinery.

It is not easy to learn the costs which are truly chargeable to the maintenance and operation of public bridges. One usually finds such items as new globes for street lamps and new pavement materials placed with items for repair timber for draw piers and repair parts of operating equipment.

The following is a brief description of the aforesaid bridges over the Willamette River, at Portland, Ore., which are operated by Multnomah County:

The Broadway Bridge, which is an electrically operated, double-leaf, Rall bascule bridge, spanning a channel width of 250 ft. When closed the vertical

clearance of this bridge above low water is 70 ft. The width of the roadways and walks is 70 ft.

The Burnside Bridge, which is a steam-operated, swing bridge, spans two channels 150 ft. wide, has a clearance of 43 ft. above low water, and is 50 ft. wide.

The Morrison Bridge, which is an electrically operated, swing bridge, spans two channels 150 ft. wide, has a clearance above low water of 36 ft. and is 55 ft. wide.

The Hawthorne Bridge, which is an electrically operated, vertical lift bridge, over a channel 240 ft. wide, has a clearance of 45 ft. above low water and is 65 ft. wide.

The data contained in Table 7 relative to the operation, etc., of these bridges, were taken from the records of the Board of County Commissioners of Multnomah County, Oregon. In this table, the letters, "BR", "B", "M", and "H", refer to the Broadway, Burnside, Morrison, and Hawthorne Bridges, respectively, in order from down stream.

TABLE 7.—Operating Data for Four Bridges Over the Willamette River, at Portland, Ore.

Date and bridge.	Number of times opened.	Total number of minutes open.*	Maximum time of any opening, in minutes.*	Minimum time of any opening, in minutes.*	Average time per opening, in minutes.*	Electric power used, in kilowatts.
Oct., 1915, BR	104	474	11	3	4.65	6 060
B	710	3 334	15	3	4.07	.....
M	629	2 109	15	2	3.35	2 130
H	297	543	9	1	1.71	910
Nov., 1915, BR	87	430	23	3	4.94	7 510
B	716	3 401	10	3	4.75	.....
M	578	1 952	12	2	3.37	2 990
H	338	579	5	1	1.71	1 060
Dec., 1915, BR	74	510	7	3	4.19	19 540
B	706	3 154	11	3	4.47	.....
M	551	1 990	10	1.5	3.61	2 700
H	424	732	8	1	1.72	1 070
Jan., 1916, BR	53	268	8	3	4.96	5 310
B	436	1 688	9	3	3.83	.....
M	492	1 827	7	2	3.30	1 317
H	335	637	5	1	1.91	960
Feb., 1916, BR	63	358	11	3	5.67	4 375
B	626	2 765	24.5	3	4.42	.....
M	499	1 845	16.5	2	3.69	2 620
H	394	735	16.5	1	1.86	1 500
March, 1916, BR	69	326	12	3	4.72	4 630
B	856	3 644	18	3	4.27	.....
M	688	2 397	18	2	3.48	2 800
H	509	881	11	1	1.73	1 170
April, 1916, BR	65	288	9	3	4.43	3 600
B	826	3 502	10.5	3	4.24	.....
M	688	2 344	8	2	3.40	2 510
H	468	702	5.5	1	1.50	1 290
May, 1916, BR	71	394	51	3	3.55	3 150
B	1 021	4 219	17	3	4.00	.....
M	832	2 841	10	2	3.41	2 280
H	618	1 017	7.5	1	1.64	880

* Time open is from gong to gong, that is, from the time the gong rings to clear the deck until it rings to open the gates to traffic, and, therefore, includes the time the span is being cleared of highway traffic.

A letter to the writer from the County Commissioners, dated November 4th, 1915, gives the operating data for the four bridges for one month, which information is contained in Table 8.

TABLE 8.

Bridge.	Number of openings.	Total time open.	Average time open.
Broadway	120	10 hrs. 37 min.	5.3 min.
Burnside	762	58 " 54 "	4.3 "
Morrison	580	37 " 17 "	3.9 "
Hawthorne	872	13 " 44 "	1.7 "

If the Morrison Bridge could have been operated with the speed of the Hawthorne Bridge, a saving of 2.2 min. per operation might have resulted, and the traffic service added to the bridge would have been $580 \times 2.2 = 1\,276$ min., or 21 hours 16 min. for the month.

The amount and value of electric current used on the bridges at the rate paid by Multnomah County for power, namely, 2.25 cents per kilowatt-hour, applied to average readings per one operation, is as follows:

Broadway Bridge	9.5 kw-hr. at 2.25 cents	= 21 cents
Harriman Bridge (lower deck)	2.2 " " " " "	= 5 "
" " (upper deck)	6.3 " " " " "	= 14 "
Morrison Bridge	5.0 " " " " "	= 11 "
Hawthorne Bridge	2.2 " " " " "	= 5 "

In conclusion, the writer desires to thank those who have discussed the paper for their additions to its value and interest; he appreciates greatly their kindly comments.

Although a number of engineers have had to do with the development of the lift bridge, this paper would be incomplete without due and proper acknowledgment that the principal credit therefor must go to John Lyle Harrington, M. Am. Soc. C. E., who took up the lift-bridge idea after it had been dormant for more than a dozen years. Bringing experience in mechanical engineering as well as in bridge design to the problem, he recreated and revived the lift bridge as a rational machine, and led the way in developing the modern designs and in securing their adoption.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

Paper No. 1479

ROLL OF HONOR

RECORD OF MEMBERS OF THE SOCIETY WHO SERVED IN THE ARMY OR NAVY OF THE UNITED STATES OR OF ITS ALLIES DURING THE WORLD WAR

The following list of members of the American Society of Civil Engineers who served in the Army, Navy, or Marine Corps of the United States or of its Allies during the World War contains only the names of those who were in active service prior to the Armistice, November 11th, 1918, and who were at some time during such service members of the Society. The information here given is abstracted from the returns of a Questionnaire mailed to all members known to have been in the service. In some cases returns were never received; these are indicated by an asterisk (*). The answers to the Questionnaire, in general, are quite complete, and it should be noted that the abstracts here given are condensed to include essential information only. The original Questionnaire returns will be preserved in the Society files as valuable records, accessible to all members who desire to refer to them.

In addition to the standard abbreviations used in the Year Book, the following military abbreviations have been adopted:

Adjutant ..... Adj.	Civil Engineer Corps ..... C. E. C.
Air Service ..... A. S.	Coast Artillery Corps ..... C. A. C.
American Expeditionary Forces ..... A. E. F.	Colonel ..... Col.
Army Transport Service .... A. T. S.	Commanding General ..... C. G.
Artillery ..... Arty.	Commanding Officer ..... C. O.
Battalion ..... Bn.	Company ..... Co.
Battalion Commander ..... Bn. Comdr.	Company Commander ..... Co. Comdr.
Brigade ..... Brig.	Corps of Engineers ..... C. of E.
Brigadier General ..... Brig. Gen.	Corporal ..... Cpl.
British Expeditionary Forces ..... B. E. F.	Engineer Officers Reserve Corps ..... E. O. R. C.
Canadian Expeditionary Forces ..... C. E. F.	Engineer Officers Training School ..... E. O. T. S.
Captain ..... Capt.	Field Artillery ..... F. A.
Chemical Warfare Service .. C. W. S.	Field Artillery Reserve Corps ..... F. A. R. C.

First Lieutenant	1st Lt.
First Sergeant	1st Sgt.
Fort	Ft.
General	Gen.
General Headquarters	G. H. Q.
General Staff	G. S.
Hospital	Hosp.
Infantry	Inf.
Lieutenant	Lt.
Lieutenant Colonel	Lt. Col.
Major	Maj.
Major General	Maj. Gen.
Master Engineer	M. E.
Member, Order of the British Empire	M. B. E.
Motor Transport Company	M. T. Co.
Motor Transport Corps	M. T. C.
National Army	N. A.
National Guard	N. G.
Ordnance Department	Ord. Dept.
Ordnance Reserve Corps	Ord. R. C.
Private	Pvt.
Quartermaster	Q. M.
Quartermaster Corps	Q. M. C.
Railway Transport Corps	R. T. C.
Regiment	Regt.
Reserve Officers Training Corps	R. O. T. C.
Sanitary Corps	San. C.
Section	Sec.
Service of Supply	S. O. S.
Signal Corps	Sig. C.
Signal Reserve Corps	Sig. R. C.
Stars	Bronze stars
Station	Sta.
Tank Battalion	Tank Bn.
Transportation Corps	T. C.
United States Army	U. S. A.
United States Marine Corps	U.S.M. C.
United States Navy	U.S.N.

DIED IN SERVICE

VANSITTART, GEORGE EDWARD. *Maj., 13th Battery, Canadian Field Artillery, 2d Canadian Div.; wounded in action (France); died May 14th, 1916.*

AGNEW, AUGUSTUS WATEROUS. *Capt., 3d Canadian Pioneers; wounded in action (France); died September 17th, 1916.*

HAGUE, WILLIAM. *1st Lt., Engr. R. C., 116th Engrs.; died in service (France), January 2d, 1918.*

GOODFELLOW, JAMES GORDON. *Maj., Royal Engrs.; killed in action (France), March 23d, 1918.*

BUCKWALTER, HARRIS DANIEL. *Capt., Engr. R. C.; killed in action (France), May 11th, 1918.*

LINDBERY, CHARLES ARTHUR. *Capt., Engr. R. C.; died in service (Camp Lee, Va.), May 27th, 1918.*

DEDICKE, ERNEST CHARLES. *1st Lt., Engrs., U. S. A.; wounded in action (France); died July 15th, 1918.*

SLEPPY, KIRBY BALDWIN. *Capt., Engrs., U. S. A., 4th Engrs.; killed in action (France), August 4th, 1918.*

ANNEAR, EDGAR HAROLD. *Capt., Engr. R. C.; died in service (Hoboken, N. J.), August 28th, 1918.*

ALEXANDER, EDWARD PORTER. *1st Lt., Engrs., U. S. A., 509th Engrs.; died in service (France), September 5th, 1918.*

PRITCHETT, FREDERICK BORRADAILE. *2d Lt., U. S. A., 109th F. A., Headquarters Co., 28th Div.; wounded in action (France); died September 6th, 1918.*

McCLURE, HUNTER. *1st Lt., Engrs., U. S. A., Co. N, 21st Engrs.; died in service (France), September 26th, 1918.*

MURRAY, JAMES POWELL. *Capt., Engrs., U. S. A.; died in service (Austin, Tex.), September 28th, 1918.*

REILLY, CHARLES GILBERT. *Capt., U. S. A., Co. D, 313th Inf.; killed in action (France), September 29th, 1918.*

DEAN, STANLEY. *Capt., Q. M. C., Constr. Div., U. S. A.; died in service (Camp Humphreys, Va.), October 2d, 1918.*

REAM, WARD HALL. *1st Lt., Engrs., U. S. A., Co. C, 305th Engrs.; killed in action (France), October 4th, 1918.*

FISKE, HAROLL LASELLE. *2d Lt., Co. C, 305th Machine Gun Bn., 77th Div.; killed in action (France), October 6th, 1918.*

PECK, MYRON HALL. *Capt., Engrs., U. S. A., 2d Engrs.; killed in action (France), October 9th, 1918.*

CHRISTOPHERS, REGINALD GILLON. *2d Lt., 34th Reinforcements, New Zealand Expeditionary Forces; wounded in action (France); died October 13th, 1918.*

HONEYMAN, BRUCE RITCHIE. *Capt., Engrs., U. S. A., Co. A, 313th Engrs.; died in service (France), October 5th, 1918.*

MARRIAN, RALPH RICHARDSON. *1st Lt., Engrs., U. S. A., Co. B, 105th Engrs.; killed in action (France), October 17th, 1918.*

MILLS, ADELBERT PHILO. *Capt., Engrs., U. S. A.; died in service (France), October 20th, 1918.*

JONES, GRANDVILLE REYNARD. *Capt., San. C., U. S. A.; died in service (Camp Benning, Ga.), December 22d, 1918.*

BELL, VICTOR HUGO. *2d Lt., Sig. C., U. S. A.; died in service (Arcadia, Cal.), January 6th, 1919.*

SLIFER, HIRAM JOSEPH. *Lt.-Col., U. S. A., 21st Engrs. (Light Ry.); died in service (France), February 3d, 1919.*

HAZLEHURST, JAMES NISBET. *Maj., Engrs., U. S. A.; died in service (France), February 9th, 1919.*

HALCOMBE, NORMAN MARSHALL. *Maj., Royal Flying Corps (British); died in service (Port Said, Egypt), February 15th, 1919.*

SERVICE RECORD

ABBOT, FREDERIC VAUGHAN
Entered service June 19, 1875; through all grades in C. of E., U. S. A., to Brig. Gen., Aug. 5, 1917. Retired from active service May 10, 1920. In command engr. replacement and special troop training camps near Washington, D. C.; Prin. Asst. to Chf. of Engrs., U. S. A.; Acting Chf. of Engrs. for five months. Organized over 225 000 engr. troops.

ABBOT, FREDERICK WILLIAM
Entered service as Pvt., British Army, Sept., 1914; 2d Lt., May, 1916; Lt. Col., Sept., 1917. Discharged Apr. 15, 1919. Member, Anglo-Russian Sub-Comm. in U. S.; Deputy-Director, Insp. Dept., Ministry of Munitions, serving in U. S.

ACHER, ALBERT HILANDS
Entered service June 15, 1905; through all grades in C. of E., U. S. A., to Col., Aug. 1918. Resigned Nov. 6, 1919. Overseas service Apr. 30, 1918–June 29, 1919. Col., 4th Engrs.; Chf. Engr., 4th Div.; Col., 102d Engrs.; Chf. Engr., 27th Div.; with U. S. Peace Comm. on evaluation of war damages. Distinguished Service Medal. Five stars.

ACKER, ROBERT LOUIS
Entered service Feb. 6, 1918, as Pvt., F. A.; Cpl., F. A., Apr. 15, 1918; 2d Lt., F. A., Mar. 31, 1919. Discharged May 2, 1919. Overseas service Apr. 23, 1918–Apr. 29, 1919. With 76th F. A., 3d Div., and 91st Div. Hdqrs. Four stars.

ACKERMAN, ALEXANDER SEYMOUR
Entered service May 8, 1917; 1st Lt., E. O. R. C., Feb. 14, 1917; Capt., Engr. R. C., Jan. 25, 1918. Discharged Aug. 9, 1919. Overseas service June 20, 1918–July 16, 1919. Constr. Div., Camps and Terminals; with Sec. Engr., Base Sec. No. 1, A. E. F.; C. O., 137th Engrs.; Acting Sec. Engr., Base Sec. No. 1, A. E. F.

ACKERMAN, ARTHUR POPE
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., June 23, 1917. Discharged Feb. 12, 1919. Overseas service July 10, 1918–Jan. 22, 1919. With 517th Engrs. in forestry work in Haute-Marne; in command saw-mill camps in advance zone, forestry troops, 1st Army, A. E. F.

ADAMS, ARTHUR
Entered service Feb. 1, 1918; Maj., Ord. Insp. Div., Jan. 24, 1918; Lt. Col., Ord. Insp. Div., Oct. 1, 1918. Discharged Aug. 8, 1919. Administration Officer, Insp. Div., Washington; Insp. Mgr., New York Ord. Office; Secy., New York Dist. Ord. Claims Board.

ADAMS, EDWARD MAGUIRE
Maj., Engrs., U. S. A.*

ADAMS, MILTON JEWELL
Entered service May 14, 1917; Capt., E. O. R. C., July 3, 1917. Discharged May 30, 1919. Overseas service Aug. 21, 1918–May 3, 1919. Co. and Bn. Comdr., 114th Engrs.

ADAMS, RAYMOND EDMOND
Entered service May 24, 1917; Capt., Q. M. C., U. S. A., Feb. 23, 1917. Discharged Feb. 5, 1919. In chg. constr. and repair, Dept. Q. M., Southeastern Dept.

ADAMS, WALTER FRANCIS
Entered service Sept. 27, 1917, as Pvt., Engrs., N. A.; Cpl., Engrs., N. A., Nov. 1, 1917; Sgt., Engrs., N. A., Feb. 20, 1918. Discharged June 27, 1919. Overseas service Mar. 30, 1918–June 16, 1919. With 23d Engrs. in railroad work in Advance Sec. and in road work with the 1st Army. Two stars.

ADAMS, WILLIAM HENRY
Entered service Aug. 8, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Apr. 16, 1919. Throughout service on staff of Chf. of Constr. Div. as Sec. Engr., Sec. E, Bldg. Branch, Constr. Div.

AGNEW, AUGUSTUS WATEROUS
Entered service 1915 as Capt. and Adj., 48th Bn., Canadian Army, afterward 3d Canadian Pioneer Bn. In chg. of constr. of trenches in Ypres Salient; Instr., Pioneer Depot, South Cæsar's Camp, Shorncliffe, England. Died Sept. 17, 1916, from wounds received in action at Courcelette, France.

ALBERT, FREDERICK WILHELM
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 19, 1917; Maj., E. O. R. C., Dec. 11, 1917; Lt. Col., N. A., May 17, 1918. Overseas service June 30, 1918–June 27, 1919. Camps Gordon and Devens in organization and preparation of engr. units for active service; with Sec. Engr., Base Sec. No. 5, A. E. F., in debarkation camp construction and installations; with Sec. Engr., Base Sec. Nos. 1 and 9; with Training Sec., Office of Chf. of Engrs., and with Congressional Comm. on reclassification of salaries. Citation for meritorious service. Merite Agricole.

ALDEN, HERBERT CLARENDEN
1st Lt., C. A. C., U. S. A.*

ALDEN, LANGFORD TAYLOR
Entered French Army in 1916. Conducteur, Service de Santé; Sgt. in ambulance service; Cannonier, Brigadier, Aspirant, Sous-Lt.; in Foreign Legion, Oct., 1917; 32d R. A. C. (F. A.); 51st R. A. C. Discharged July 6, 1919. Overseas service 1916–June, 1919. With S. S. A. served at Chemin des Dames, Verdun, Champagne, Woevre, etc.; with R. A. C. at Chemin des Dames, Verdun, Champagne, Ardennes, Vosges, etc. Croix de Guerre with two citations. Twenty-one stars (French.)

ALDERMAN, ERNEST SAMUEL
Entered service Sept. 12, 1918, as 2d Lt., Q. M. C., Constr. Div., U. S. A. Discharged Apr. 9, 1919. Officer in chg. roads and grounds, Camp Travis.

ALEXANDER, EDWARD PORTER
Entered service 1917 as 1st Lt., E. O. R. C. Overseas service, 1918. E. O. T. C., Fort Snelling, Fort Leavenworth and Fort Travis; Adj., 509th Engrs. Died Sept. 5, 1918, in active service at St. Nazaire, France.

ALEXANDER, KAY
After enlistment in 239th Overseas Ry. Constr. Corps, Canada, was appointed to commissioned rank Sept. 12, 1916. Discharged as Maj., Canadian Ry. Troops, Mar. 19, 1919. Served in Canada, England, and France with 239th Overseas Ry. Constr. Corps; 3d and 12th Bns., Canadian Ry. Troops; constr. and maintenance of light rys. along British front from Ypres to Amiens. Officer of the Order of the British Empire.

ALLEN, FRANKLIN REA
Entered service Aug. 25, 1917; Capt., Engrs., N. A., Aug. 1, 1917. Discharged July 17, 1919. Overseas service Aug. 25, 1918–June 20, 1919. Ft. Leavenworth; Instr., Camp Lee; with 603d Engrs. One star.

ALLEN, HERSCHEL HEATHCOTE
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., June 23, 1917. Capt., Engr. R. C., Dec. 11, 1917. Discharged Jan. 17, 1919. Instr., E. O. T. S.; Bn. Adj., 214th Engrs.

ALLEN, RALPH BENJAMIN
Entered service May 12, 1917; 1st Lt., Engrs., N. A., Aug. 15, 1917. Discharged June 2, 1919. Overseas service Oct. 31, 1917–May 12, 1919. Co. Comdr., 25th Engrs.; in chg. of levee constr. at Is-sur-Tille; in chg. of constr., 1st Army H. Q., Ligny-en-Barrios.

ALLEN, RAYMOND CLEAVELAND
Entered service Aug. 24, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Asst. to Constr. Q. M., and Utilities Officer, Boston Army Base; Constr. Q. M., Coast Defense, Boston.

ALLEN, WALTER HENRY
Maj., Engrs., U. S. A.*

ALLEN, WALTER HINDS
Entered service June 27, 1903; through all grades in C. E. C., U. S. N., to Comdr., July, 1918. Asst. Public Works Officer, and Public Works Officer, Navy Yard, New York; Public Works Officer, Great Lakes Naval Training Station.

ALLISON, LEONIDAS METCALFE
Entered service Apr. 26, 1918; 2d Lt., Inf., U. S. A., Aug. 26, 1918. Discharged Dec. 2, 1918. 2d Inf. Replacement Regt., Camp Gordon; M. T. C., Camp Johnson.

ALLISON, WILLIAM FRANKLIN
Maj., Engrs., U. S. A., A. E. F.*

ALTMAN, FRANK STORK
Entered service May 8, 1917, as 2d Lt., E. O. R. C.; 1st Lt., Engr. R. C., Jan. 18, 1918; Capt., Engrs., U. S. A., Sept. 8, 1918. Discharged Aug. 5, 1919. Overseas service Mar. 30, 1918–July 9, 1919. Co. Comdr., 23d Engrs.; C. O., Motor Reception Park, Havre, France; C. O., 2d Provisional Water Train, St. Mihiel offensive; C. O., 1st Provisional Gasoline Train, Meuse-Argonne offensive. Two stars.

ALTSTAETTER, FREDERICK WILLIAM
Entered service June, 1893; Lt. Col., C. of E., U. S. A., at beginning of war; temporary Col., Engrs. In chg. Engr. Dist., H. Q. at Grand Rapids and Detroit; river and harbor work.

ALVEY, JAMES PERRIE, JR.
Entered service Mar. 20, 1918, as 1st Lt., Q. M. C., Constr. Div., N. A.; Capt., Engrs., U. S. A., May 16, 1919. Discharged July 1, 1919. Overseas service July 12, 1918–June 26, 1919. 157th Depot Brig., Camp Gordon; C. O., Timber Service Project No. 19, A. E. F.; Deputy and Sec. Engr., Base Sec. No. 4, A. E. F. One star.

ANDERS, FRANK LA FAYETTE
Entered service Sept. 11, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Oct. 21, 1919. Asst. to Officer in Chg. Utilities, Camp Dodge; Officer in Chg. Utilities, Gen. Hosp. No. 28, Fort Sheridan.

ANDERSEN, CHRISTIAN
Entered service Aug. 9, 1918, as Capt., Ord. Dept., U. S. A. Discharged Jan. 15, 1919. In Eng. Div., Ord. Dept., Washington, D. C.

ANDERSON, CHARLES LOUIS BATES
Lt. Comdr., C. E. C., U. S. N. R. F.*

ANDERSON, JOHN EDWARD
Entered service Jan. 5, 1915, as 2d Lt., Royal Engrs., Great Britain; Lt., Royal Engrs., Sept., 1915; Capt., Royal Engrs., Sept. 1917; Maj., Royal Engrs., Oct. 1, 1918.* Overseas service Sept. 10, 1914, to date. 11th Co. (Field), 2d Div., B. E. F.; Adj., 33d Div. Engrs.; C. O., 212th Co. (Field), Royal Engrs.; Dist. Engr., Railways, Baghdad, Mesopotamia. Mentioned in Sir John French's and Sir Douglas Haig's despatches; Military Cross and Bar; Croix de Guerre, with Palm.

ANDERSON, JOHN HENNING
Entered service July 20, 1917, as 1st Lt., Engrs., N. A.; Capt., Engrs., N. A., Dec. 10, 1917. Discharged Jan. 17, 1919. Regtl. Adj., 212th Engrs.

ANDERSON, WALTER SEIGFREID
Entered service Jan. 28, 1918, as Cadet, School of Military Aeronautics, Ohio State Univ. Discharged Dec. 16, 1918. School of Fire, A. S., Fort Sill, Okla.

ANDERSON, WILLIAM POPE
Entered service Sept. 26, 1917; Capt., Engrs., N. A., July 30, 1917. Discharged Apr. 11, 1919. Overseas service Jan. 26, 1918–Feb. 6, 1919. In training with 112th Engrs.; with G–1, G. S., A. E. F.; Motor Dispatch Service, A. E. F.; Financial Engr. Officer, Base Sec. No. 3, S. O. S., A. E. F.

ANDREW, CLARENCE RAYMOND
Entered service Sept. 26, 1917; Capt., E. O. R. C., Aug. 1, 1917. Discharged July 30, 1919. Overseas service June 15, 1918–June 9, 1919. Co. Comdr., 32d Engrs., in chg. ry. ballast work, St. Sulpice, France; C. O., U. S. troops, Talmont, France.

ANDREWS, CARL BOWERS
Entered service June 15, 1918; Capt., E. O. R. C., July 31, 1917. Discharged Dec. 28, 1918. Camps Lee, Fremont, and Humphreys as student and Instr.

ANDREWS, JAMES HENRY MILLAR
Entered service July 15, 1917, as Maj., Engrs., N. A.; Maj., Ord. Dept., N. A., Feb. 6, 1918; Lt. Col., Ord. Dept., U. S. A., Oct. 5, 1918. Discharged Oct. 16, 1919. Bn. Comdr., 103d Engrs.; detached service, Office Chf. of Ord., Washington, D. C.; C. O., Raritan Arsenal.

ANGWIN, HENRY RAYMOND
Entered service July, 1918, as Pvt., 1st Engr. Replacement Regt. Discharged Nov., 1918. Student, E. O. T. S.

ANNEAR, EDGAR HAROLD
Entered service Jan., 1918, as Capt., E. O. R. C.; Maj., Engrs., U. S. A., 1918. Overseas service, 1918. E. O. T. C., Camp American University; Co. Comdr., 43rd Engrs. Died Aug. 28, 1918, in active service at Hoboken, N. J.

APPLEGARTH, GAULT
Entered service May 12, 1917, as Pvt., E. O. R. C.; 2d Lt., E. O. R. C., July 10, 1917; 1st Lt., E. O. R. C., Aug. 15, 1917; Capt. Engr. R. C., Dec. 1, 1917. Overseas service Mar. 29, 1918–June 12, 1919. Discharged July 10, 1919. With 23d Engrs., road building, Meuse-Argonne and St. Mihiel offensives. Two stars.

ARAKAWA, FUTOSHI
Entered service June 1, 1918. Discharged July 23, 1919. With 2d Inf.

ARDERY, EDWARD DAHL
Entered service June 16, 1902; through all grades in C. of E., U. S. A., to Maj., May 15, 1917; Lt. Col., Aug. 5, 1917; Col., Engrs., N. A., June 19, 1918. Overseas service Oct. 3, 1917–Sept. 19, 1918. Returned to permanent rank as Maj., C. of E., U. S. A., Mar. 15, 1920. Asst. to Chf., C. W. S., A. E. F.; Chf. Gas Officer, 2d Corps; Chf. Gas Officer, 1st Army; in chg. organization of engr. troops, Camp Forrest; Office, Chf. of Engrs., Washington, D. C. One star.

ARMITAGE, GEORGE WASHINGTON
Entered service Oct. 26, 1917; Capt., Q. M. C., N. A., Oct. 3, 1917; Maj., Q. M. C., Constr. Div., U. S. A., Oct. 1, 1918. Asst. to Dept. Q. M., Hawaiian Dept.; in chg. constr. and repair, Port of Embarkation, New York.

ARMS, LEO MURRY
Entered service Sept. 25, 1917; 2d Lt., Engr. R. C., Aug. 8, 1917; 1st Lt., Engrs., U. S. A., July 30, 1918. Overseas service Feb. 1, 1918–Aug. 20, 1918. Discharged Nov. 30, 1918. With 308th Engrs., Camp Sherman; engr. replacement depot, France; 2d Engrs.; 4th, 7th and 8th Engr. Training Regts., Camp Humphreys; 97th Engrs.

ARMSTRONG, CHARLES JOHNSTONE
Entered service June 27, 1893; Lt. Col., Canadian Engrs., Sept. 3, 1914; Brig. Gen., Canadian Engrs., Sept. 13, 1915. Overseas service Sept. 22, 1914–June 28, 1919. Chf. Engr., Canadian Army Corps; Commanding Canadian Engrs., 1st Canadian Div.; Acting Gen. Officer commanding Dist. No. 4, Montreal; Chf. Engr., Cavalry Corps, British Army; Chf. Engr., 7th British Corps. Companion of the Order of St. Michael and St. George; Companion of the Order of the Bath; Grand Officer, Military Order of Avis, Portugal; Commander of the Order of the Crown, Belgium; mentioned four times in despatches.

ARMSTRONG, HARRY ARTHUR
Entered service July 2, 1918, as Pvt., E. O. T. C., Camp Humphreys; 2d Lt., Engrs., U. S. A., Nov. 15, 1918. Discharged Dec. 11, 1918. With 149th Engrs.

ARMSTRONG, MERWIN
Entered service May 8, 1917; Capt., E. O. R. C., June 17, 1917. Overseas service May, 1918–Feb. 17, 1919. Discharged Feb. 19, 1919. 105th Engrs.; Acting Stores Officer, Engr. Supplies, 30th Div., with British 2d Army in Belgium; with British 4th Army on Somme. Two stars. Wounded Sept. 29, 1918, at Bellicourt.

ARMSTRONG, ROGER WELLINGTON
Capt., Q. M. C., U. S. A.*

ARN, WILLIAM GODFREY
Entered service May 8, 1917, as Capt., E. O. R. C.; Maj., Engr. R. C., Mar. 1918; Lt. Col., Engrs., U. S. A., Apr., 1919. Overseas service July 21, 1917–Apr. 28, 1919. Discharged June 1, 1919. With 13th Engrs., in chg. of M. of W. and structures, Verdun Sector. Citation from Gen. Pershing for meritorious service. Three stars.

ARNOLD, BION JOSEPH
Entered service Jan. 23, 1917, as Maj., E. O. R. C.; Lt. Col., Sig. C., A. S., U. S. A., Dec. 14, 1917. Discharged Feb. 16, 1919. Active in formulating Officers Reserve Corps Act; in command development of aerial torpedo; consultant for U. S. Shipping Board, Port Facilities Commission, Washington, D. C.

ASHKINS, NATHAN THOMAS
Entered service Apr. 7, 1917, as 2d Lt., E. O. R. C.; 1st Lt., Tank C. Overseas service Apr. 15, 1917–Mar. 21, 1919. Discharged Apr. 26, 1919. With French Army in Aisne and Champagne offensives; with Director Gen., Transportation, A. E. F.; 306th Tank Bn.

ASHLEY, CARL
Entered service Aug. 11, 1917; Capt., E. O. R. C., July 29, 1917; Maj., T. C., Oct. 31, 1918. Overseas service Aug. 18, 1917-Oct. 20, 1919. Discharged Oct. 31, 1919. With Engr. of Constr., T. C.; in chg. inland water transport at Paris.

ASHMEAD, PERCY HERBERT†
Maj., Engrs., U. S. A.*

ASHTON, RAYMOND
Entered service Oct. 31, 1917, as Pvt., 1st Class, Sig. R. C.; 2d Lt., A. S., U. S. A., May 15, 1918. Overseas service July 7, 1918–Jan. 8, 1919. Discharged Jan. 20, 1919. Engr. Officer at aviation training stations in France; Instr., Aerial Navigation, St. Maixent, France.

ASHWORTH, FRANK KARR
1st Lt., Engrs., U. S. A.*

ASPLUNDH, EDWIN THEODORE
Entered service May 3, 1917; 1st Lt., E. O. R. C., June 11, 1917; Capt., E. O. R. C., Aug. 4, 1917. Overseas service May 28, 1918–Apr. 25, 1919. Discharged Apr. 28, 1919. Supply Officer and Co. Comdr., 103d Engrs.

ASSERSON, HENRY RAYMOND
Entered service Sept. 29, 1917; Maj., E. O. R. C., June 24, 1917. Overseas service Oct. 9, 1917–Feb. 17, 1919. Discharged Feb. 20, 1919. On staff, Chf. Engr., Line of Communications, A. E. F.; in chg. constr. work, and Engr. Disbursing Officer, Gievres, France.

ATTERBURY, WILLIAM WALLACE
Entered service Sept. 14, 1917; Brig. Gen., T. C., Oct. 4, 1917. Overseas service Sept. 14, 1917–May 31, 1919. Discharged June 1, 1919. Director Gen. of Transportation, A. E. F., throughout service. Distinguished Service Medal; Commandeur, Legion d'Honneur; Companion of the Order of the Bath; Commander of the Order of the Crown, Belgium.

ATWOOD, WILLIAM GREENE
Entered service June 11, 1917, as Maj., E. O. R. C.; Lt. Col., Engr. R. C., Feb. 25, 1918; Col., Engrs., U. S. A., Feb. 26, 1919. Overseas service July 28, 1917–Sept. 27, 1919. Discharged Aug. 18, 1919. C. O., 17th Engrs.; Asst. to Sec. Engr., and Sec. Engr., Base Sec. No. 1, A. E. F.; Head of Transportation Dept. for Herbert C. Hoover. Citation for meritorious service from G. H. Q., A. E. F.; Officier, Legion d'Honneur; Gr. II, Order of St. Sava, Serbia.

AUSTILL, HURIBOSCO
Entered service Sept. 2, 1917; Capt., E. O. R. C., May 23, 1917; Maj., Engrs., N. A., July 30, 1918. Overseas service Nov., 1917–Sept., 1918. Discharged Dec. 13, 1918. Co. Comdr., 501st Engrs.; in chg. constr., Ord. Base Depot, France.

AUSTIN, HERBERT ASHFORD ROBERTSON
Entered service June 15, 1918; 1st Lt., Engrs., N. A., Sept. 4, 1917. Discharged Dec. 17, 1918. 3d and 9th Engr. Training Regts.

AYRES, JOHN HENRY
Entered service Mar. 16, 1918, as Capt., Engrs., N. A. Overseas service May 10, 1918–Jan. 13, 1919. Discharged Jan. 23, 1919. With 42d Engrs.

AYRES, QUINCY CLAUDE
Entered service May 3, 1917; 2d Lt., E. O. R. C., Feb. 4, 1917; 1st Lt., Engrs., U. S. A., May 8, 1919. Overseas service Dec. 3, 1917–June 9, 1919. Discharged June 30, 1919. Engr. Instr.; 1st, 2d, and 21st Engrs.

BABBITT, HAROLD EATON
Entered service May 15, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service Nov. 24, 1917–Dec. 25, 1918. Discharged Jan. 10, 1919. Engr. Instr.; Office Sec. Engr., Advance Sec., S. O. S., A. E. F.

BAFFREY, CHARLES RAYMOND
Entered service Aug. 16, 1914, as 2d Lt., F. A., French Army; 1st Lt., F. A., Sept. 30, 1914; Capt., F. A., May 27, 1917. Discharged Mar. 1, 1919. With 22d F. A., 106th Regt. and 102d Regt., French Army. Regtl. and divisional citations. Chevalier, Legion d'Honneur. Wounded in neck with shrapnel.

BAILEY, LEWIS PENN
Entered service May 7, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service July 9, 1918–May 29, 1919. Discharged July 28, 1919. Co. Comdr., 304th Engrs.; Regtl. Personnel Adj.; Bn. Adj.

BAILHACHE, JOHN GOODIN
2d Lt., Engrs., U. S. A.*

† Died Nov. 11, 1919.

BAKENHUS, REUBEN EDWIN
Entered service Feb. 27, 1901; Comdr., C. E. C., U. S. N., July 1, 1917; Capt., C. E. C., U. S. N., Mar. 24, 1919. Project Mgr., Bureau of Yards and Docks, Navy Armor and Projectile Plants; Asst. Mgr., and Mgr., Ship Yard Plants, Emergency Fleet Corp.; Asst. Chf., Bureau of Yards and Docks, Navy Dept., Washington, D. C. Navy Cross.

BAKER, GEORGE LIVINGSTON
Entered service May 20, 1918, as 1st Lt., Inf., U. S. Guards; Capt., U. S. Guards, July 20, 1918; Capt., Inf., U. S. A. Discharged Apr. 8, 1919. U. S. Guards, Ft. Niagara, N. Y.; Military Police, Camp Dix; 22d Inf., Ft. Jay, N. Y.

BAKER, HORACE SINGER
Entered service July 19, 1917, as Capt., Engrs., N. A.; Lt. Col. Engrs., N. A., Nov. 18, 1917; Col., Engrs., U. S. A., Sept. 3, 1918. Overseas service, July 18, 1918–May 30, 1919. Discharged July 7, 1919. Const. Q. M., Camp Bowie; with 111th Engrs. Two stars.

BAKER, SHIRLEY
Entered service Sept. 1, 1917, as Capt., E. O. R. C.; Maj., Engrs., U. S. A., Mar. 13, 1919. Overseas service Jan. 22, 1918–June 9, 1919. Discharged June 28, 1919. Co. Comdr., 23d Engrs.; Asst. Army Roads Officer, 1st Army, during St. Mihiel and Meuse-Argonne offensives. Two stars.

BALCH, WILLIAM HOYT
Entered service Sept. 1, 1917; Capt., E. O. R. C., May 17, 1917; Maj., Engrs., U. S. A., Nov. 8, 1918. Overseas service, Oct. 30, 1917–Apr. 29, 1919. Discharged May 19, 1919. With Dept. of Light Rys. and Roads. Citation from Gen. Pershing for meritorious service; Officier, Order de l'Etoile Noire.

BALDWIN, THOMAS ABBOTT
Entered service Mar. 30, 1918, as Lt., U. S. N.; Lt. Comdr., U. S. N., June 20, 1919. Overseas service June 10, 1918-Feb. 28, 1919. Released from active duty Aug. 1, 1919. In chg. erection of steel towers for radio station at Croix d'Huis, France.

BANDY, EDWARD LEE
1st Lt., Engrs., U. S. A.*

BARBER, CHARLES WIGHTMAN
1st Lt., Engrs., U. S. A., A. E. F.*

BARBER, JUSTIN FREDERIC
Entered service Nov. 27, 1917, as Pvt., Engrs., N. A.; Sgt., 1st Class, Engrs., N. A., Dec. 28, 1917. Overseas service Mar. 31, 1918–June 9, 1919. Discharged June 24, 1919. With 23d Engrs., road maintenance at Libourne, France, and in the Meuse-Argonne Sector. One star.

BARBER, NORMAN NATHANIEL
Entered service June 13, 1917, as 1st Lt., E. O. R. C.; Capt., Engrs., N. A., Feb. 26, 1918. Overseas service Aug. 1, 1917–May 4, 1919. Discharged May 12, 1919. With 16th and 302d Engrs., Meuse-Argonne offensive. Cited in divisional orders, 77th Div.; certificate from Gen. Pershing for gallantry in action. One star.

BARCLAY, ARTHUR JACKSON
1st. Lt., Engrs., U. S. A.*

BARNARD, ARCHER FORTESCUE
Entered service Aug. 29, 1918, as 1st Lt., Engrs., U. S. A. Discharged April 16, 1919. In office of Director Gen. of Military Rys., under Chf. of Engrs., U. S. A.

BARNEY, SAMUEL EBEN
Entered service Aug. 8, 1917, as Maj., Engrs., N. A. Released from active service Sept. 10, 1917. Instr. in Eng. Sec. of the R. O. T. C. at Yale Univ.

BARRY, EDMUND JOSEPH
Entered service Feb. 14, 1918, as Maj., Engrs., N. A. Const. Q. M., Bag Loading Plant at Tullytown, Pa.; Mexican Border Project; in chg. survey and purchase, Camp Dix; Const. Q. M. and Utilities Officer, Ft. Leavenworth.

BARSTOW, EUGENE DUSTON
Entered service Jan. 5, 1918; 1st Lt., Engrs., N. A., Sept. 16, 1917. Discharged Feb. 16, 1918.

BARTHOLOMEW, BRADLEY WHITE
Entered service May 6, 1917; 2d Lt., E. O. R. C., June 23, 1917; 1st Lt., Engr. R. C., May 28, 1918. Overseas service July 14, 1918–June 13, 1919. Discharged July 7, 1920. With 301st Engrs.; Toul Sector and Army of Occupation. One star.

BASCOM, GEORGE ROCKWELL
Entered service Jan. 8, 1918; Maj., San. C., U. S. A. Discharged Mar. 12, 1920. Camp San. Engr., Camp Pike; special san. engr. work in about 20 camps; in chg. San. Eng. Sec.

BASS, FRED THOMSON
Entered service Dec. 21, 1917, as Capt., Engrs., N. A. Overseas service June, 1918–June, 1919. Regtl. Adj., 32d Engrs.; Aide to Maj. Gen. George Bell, Jr.

BASSETT, HERBERT HOWARD
Entered service Feb. 25, 1918; Maj., Engrs., N. A., Feb. 22, 1918. Discharged Sept. 10, 1919. Asst. to Officer in Chg. Cantonment Div. (later Chf. of Constr. Div.); Const. Q. M., Emergency Fleet Housing Project; Asst. Const. Q. M., Gen. Hosp. No. 3 and Const. Q. M., Gen. Hosp. No. 35.

BATTIE, HERBERT SCANDLIN
Entered service Sept. 25, 1917; 1st Lt., Engrs., N. A., Dec. 10, 1917. Discharged Jan. 22, 1919. With 23d and 305th Engrs.; 3d Training Regt., misc. camp constr., Camp Humphreys; 213th and 212th Engrs., Camp Forrest.

BAVER, WALTER SAMUEL
Entered service Oct. 24, 1918, as 1st Lt., San. C., U. S. A. Camp San. Engr., Call Field, Wichita Falls, Tex., Aberdeen Proving Grounds, Md., and Camps Humphreys, Upton, and Benning; Camp San. Insp. and Med. Property Officer, Army Balloon School, Ft. Omaha, Nebr.

BAXTER, ORA GROVER
Entered service Sept. 17, 1917, as Capt., Engrs., N. A. Overseas service June 30, 1918–May 23, 1919. Discharged June 7, 1919. Co. Comdr. 601st Engrs.; Chf. Instr., Army Engr. School, A. E. F. Topographic Officer, 108th Engrs.

BAYLISS, PAUL
Entered service Sept. 25, 1917; 2d Lt., Engrs., N. A., Aug. 21, 1917. Discharged Feb. 2, 1919. With 8th Engrs.

BEACH, LANSING HOSKINS
Entered service July 1, 1878; through all grades in C. of E., U. S. A., to Maj. Gen. and Chf. of Engrs., Jan. 9, 1920.

BEALE, ALLAN SOUTHER
Entered service Dec. 12, 1917, as 1st Lt., A. S., N. A. Discharged Dec. 12, 1918. Graduated from ground school; qualified as pursuit pilot.

BEALL, PENDLETON
Entered service June 11, 1917, as Pvt., Inf., N. Y. N. G.; Cpl., Inf., Apr. 18, 1918; Sgt., Inf., July 15, 1918. Overseas service Oct. 28, 1917–Feb. 17, 1919. Discharged Mar. 5, 1919. With 165th Inf. at Luneville, Baccarat, Chasseurs, Champagne, Chateau Thierry, St. Mihiel, France. Citation for gallantry in action. Four stars. Wounded once.

BEAM, CARL EUGENE
Entered service Oct. 4, 1917; 2d Lt., Engrs., N. A., June 17, 1918; 1st Lt., Engrs., U. S. A., June 28, 1919. Overseas service Sept. 20, 1918–July 23, 1919. Discharged Aug. 5, 1919. With 3d Engr. Training Regt., 541st Engrs.; Bn. Personnel and Supply Officer; Bn. Adj., with Sec. Engr., Base Sec. No. 1, A. E. F.

BEAN, PAUL JONES †
Civ. Engr., U. S. N.*

BEARD, ROBERT STANLEY
Entered service Oct. 23, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Asst. Superv. Const. Q. M. on four ord. projects; Utilities Officer, Camp Lee.

BEBB, EDWARD CROSBY
Capt., Engrs., U. S. A.*

BECK, EDWARD ADAM
Entered service Sept 10, 1918, as Capt., San. C., U. S. A. Discharged Dec. 20, 1918. Camp San. Engr., Camp Shelby.

BEE, CHARLES EVERETT
1st Lt., Engrs., U. S. A.*

BEEBE, JAMES WILBUR
Entered service Oct., 1918, as Capt., Engrs., U. S. A. Discharged Sept., 1919. Educational service, Physical Reconstruction Div., Surgeon General's Office.

BEEMAN, THOMAS RUPE
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service Nov. 2, 1917–Aug. 20, 1919. Discharged Sept. 3, 1919. With Director Gen. of Transportation, A. E. F., in Dept. of Light Rys. and Roads; with Chf. of Road Service, 2d French Army; Roads Officer under Chf. Engr., 1st Army, during Champagne-Marne and Aisne-Marne operations; 5th Army Corps Roads Officer at St. Mihiel; Insp. of Rys. and Roads, 1st Army Area, and 3d Army, A. E. F.; on Engr. Research Comm. Four stars.

BEEMER, JOHN ARTHUR
Capt., Engrs., U. S. A.*

BEERBOWER, DUMONT
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., June 19, 1917; Capt., Engr. R. C., May 22, 1918. Discharged Dec. 4, 1918. Instr., E. O. T. S.; Co. Comdr. with various units.

BEGG, ROBERT BURNS HALDANE
Entered service May 8, 1917; Capt., E. O. R. C., Feb., 1917. Overseas service Oct. 3, 1917–Mar. 6, 1919. Discharged Mar. 27, 1919. In office of Sec. Engr. and officer in chg. water supply and sewerage, Base Sec. 3, A. E. F., London.

BEHRMAN, ISADORE ELLIS
Entered service May 8, 1917, as Pvt., E. O. R. C.; 1st Lt., E. O. R. C., June 25, 1917; Capt., Engrs., U. S. A., Sept. 7, 1918; Maj., Engrs., U. S. A., Mar. 11, 1919. Overseas service Sept. 15, 1917–Oct. 1, 1919. Discharged Oct. 27, 1919. Pvt. at Ft. Myer; with 303d Engrs.; Advance Sec., A. E. F.

† Died Jan. 25, 1919.

BELL, GEORGE EDWARD
Entered service Feb. 7, 1917, as Lt., Canadian Engrs.; Capt., Canadian Engrs., Jan. 1, 1918. Overseas service Feb. 7, 1917–Apr. 5, 1919. Discharged Apr. 5, 1919. British War Office and Air Ministry; Resident Engr. on airdrome constr., England.

BELL, HARRY WALTON
Entered service May, 1918, as Capt., E. O. R. C. Overseas service Sept., 1918–July, 1919. Discharged July 2, 1919. With 22d Engrs. in light ry. work in Toul Sector. One star.

BELL, VICTOR HUGO
Entered service 1917, as Pvt., Sig. C., U. S. A.; M. S. E., Meteorological Detachment, Sig. C., U. S. A.; 2d Lt., Sig. C., U. S. A., 1918. With Science and Research Div., Army Balloon School, Arcadia, Cal. Died Jan. 6, 1919, in service at Arcadia, Cal.

BELLINGER, LYLE FREDERICK
Entered service Jan. 12, 1901, as Lt., Jr. Grade, C. E. C., U. S. N.; Lt., Apr., 1906, and Lt. Comdr., Jan., 1911. Philippine service; Navy Yards at Portsmouth, N. H., Newport, R. I., Brooklyn, Philadelphia, New Orleans, Bremerton, Wash., and Cavite, P. I.

BELZNER, THEODORE
Entered service May 8, 1917; 1st Lt., E. O. R. C., Feb. 19, 1917. Discharged Apr. 12, 1918.

BENHAM, CLAUDE GILBERT
Entered service Mar. 1, 1917, as 2d Lt., E. O. R. C.; 2d Lt., C. A. C., U. S. A., June 14, 1917; 1st Lt., C. A. C., June 14, 1917; Capt., C. A. C., Aug. 5, 1917. Overseas service Oct. 7, 1918, to date. Returned to permanent rank Mar. 16, 1920. Fort Comdr., Ft. Monroe; 4th Trench Mortar Bn., C. A. C., American Embarkation Center; Asst. Sec. Judge Advocate, St. Nazaire, France.

BENHAM, WEBSTER LANCE
Entered service Oct. 25, 1917, as Maj., Q. M. C., N. A. Discharged Apr. 30, 1919. Officer in Chg. Utilities, and Const. Q. M., Camp Funston; Regional Supervising Utilities Officer in camps of Middle West.

BENNETT, JOHN WALTER FRINK
Entered service June 5, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged Apr. 30, 1919. With Const. Q. M., Brooklyn Army Supply Base.

BENNISON, ERNEST WILLIAM
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., June 16, 1917; Capt., Engrs., U. S. A., May 9, 1919. Overseas service June 8, 1918–June 8, 1919. Discharged July 9, 1919. With 32d Engrs. on ry. and storage yard constr., St. Sulpice, France.

BENSEL, JOHN ANDERSON
Entered service May 16, 1917, as Maj., E. O. R. C. Discharged Jan. 3, 1919. On duty Norfolk, Va.; C. O., 125th Engr. Bn.

BENSON, ORVILLE
Entered service Oct. 12, 1917; Maj., E. O. R. C., June 23, 1917. Overseas service Oct. 27, 1917–Feb. 12, 1919. Discharged Oct. 7, 1919. With T. C.; Observer with 8th French Army, Toul Sector; with Director-Gen. of Transportation; Chf. Engr. Ry. Arty. Four stars.

BERDEAU, RAY WILLIAM
Entered service Sept. 5, 1917; 1st Lt., Engrs., N. A., Aug. 17, 1917; Capt., Engrs., N. A., Dec. 9, 1917. Discharged June 16, 1919. Asst. to Div. Engr., 82d Div.; Personnel Adj., 5th Training Regt., Camp Humphreys; Asst. to Div. Engr., 20th Div. Citation from Chf. of Engrs. for efficient service.

BERGAN, THOMAS BERNARD
Entered service June 8, 1918; 2d Lt., Engr. Section, San. C., N. A., May 28, 1918; 1st Lt., San. C., U. S. A., Nov. 4, 1918. Discharged Dec. 6, 1918. In chg. sanitation, drainage, water supply and sewage disposal, Camp Gordon.

BESSEY, ROY FREDERIC
Entered service June 18, 1918, as 2d Lt., E. O. R. C.; 1st Lt., Engrs., U. S. A., May 7, 1919. Overseas service June 30, 1918–Aug. 13, 1919. Discharged Aug. 22, 1919. With 57th and 126th Engrs., as Adj., Const. Officer, and Executive Officer. Citation from C. in C., A. E. F., for meritorious service.

BEST, BYRON GRAY
Entered service Nov. 5, 1918, as Pvt., F. A., U. S. A. Discharged Dec. 6, 1918. With 15th Observation Battery.

BETTS, CLIFFORD AULL
Entered service Aug. 27, 1917; 1st Lt., C. A. C., N. A., Nov. 27, 1917; Capt., C. A. C., U. S. A., Sept. 21, 1918. Overseas service Dec. 10, 1917–Nov. 27, 1918. Discharged Dec. 10, 1918. With 44th Artillery; Army Heavy Artillery School, France; Provisional Howitzer Regt.; active duty at front seven months; in Champagne with French 4th Army; 1st and 2d American Armies in St. Mihiel and Woevre operations.

BICKEL, GEORGE ROBERT
Entered service Aug. 26, 1918, as Chf. Machinist's Mate, U. S. N. Released from active duty Feb. 21, 1919. With C. E. C., U. S. N., Bureau Yards and Docks. Navy Yard, Charleston, S. C.

BIDDLE, JOHN
Maj. Gen., U. S. A.*

BILLINGS, ASA WHITE KENNEY
Entered service May 11, 1917; Lt., Sr. Grade, C. E. C., U. S. N. R. F., May 8, 1917; Lt. Comdr., Oct. 24, 1918; Comdr., July 16, 1919. Overseas service Nov. 3, 1917–Mar. 11, 1919. Released from active duty Mar. 11, 1919. Sr. Asst. to Public Works Officer, Brooklyn Navy Yard; Asst. Public Works Officer, Comdr., Naval Aviation Forces in Foreign Service; Public Works Officer, Aid for Aviation, U. S. Naval H. Q., London. Navy Cross; Legion d'Honneur.

BILLINGS, FRED MERRITT
Entered service Dec. 28, 1917; Capt., E. O. R. C., July, 1917. Discharged Aug. 6, 1919. With Constr. Div.; Const. Q. M. at Camp Kearny, Ft. Rosecrans, Camp Tecate, Camp Campo, and other California camps.

BINCKLEY, GEORGE SYDNEY
Entered service Aug. 20, 1917; Maj., E. O. R. C., Mar. 1, 1917. Overseas service July, 1918–July, 1919. Discharged Oct. 31, 1919. With 112th Engrs.; C. O., 519th Engrs., U. S. and France; Deputy Sec. Engr. and Sec. Engr., Base Sec. No. 4, A. E. F.

BINGER, WALTER DAVID
Entered service Jan. 5, 1918; 2d Lt., Aviation Sec., Sig. C., U. S. A., Jan. 16, 1919. Overseas service Feb. 8. 1918–Jan. 3, 1919. Discharged Jan. 8, 1919. Constr. Officer, 468th Aero Constr. Squadron in France; Asst. to Chf. Constr. Officer, 2d Aviation Instr. Center, also Acting Chf. Constr. Officer.

BIRDSEYE, CLAUDE HALE
Entered service June 18, 1917; Capt., E. O. R. C., Mar. 17, 1917; Maj., E. O. R. C., July 25, 1917; Lt. Col., C. A. C., U. S. A., Aug. 10, 1918. Overseas service Aug. 13, 1917–Jan. 22, 1919. Discharged June 23, 1919. 1st Brig., C. A. C.; Office, Chf. of Artillery, A. E. F., as topographic officer. Officier de l'Instruction Publique, Order of Univ. Palms. Four stars.

BISHOP, ROY PRENTICE
Entered service Sept. 2, 1917; 2d Lt., Engrs., N. A., July 30, 1917; 1st Lt., T. C., Feb. 14, 1919. Overseas service Jan. 29, 1918–May 11, 1919. Discharged May 24, 1919. With 35th Engrs., as Co. Comdr. and Personnel Officer.

BISSELL, CLINTON TALCOTT
Entered service Nov. 18, 1918; Capt., Q. M. C., Constr. Div., U. S. A., Nov. 9, 1918. Discharged Apr. 1, 1919. Asst. to Chf. of Constr. Div. in chg. fire protection at all army posts, camps, and cantonments.

BITHER, TOM ALLEN
Entered service Aug. 27, 1917, as 2d Lt., Sig. C., Aviation Sec.; 1st Lt., A. S., U. S. A., Aug. 21, 1918. Discharged Jan. 27, 1919. In training San Francisco, Rockwell Field, and March Field.

BIXBY, WILLIAM HERBERT
Entered service 1869. Returned to active duty May, 1917, after having been retired as Brig. Gen., Chf. of Engrs., U. S. A. Released from active duty Apr. 1919. River and harbor duty at Kansas City, Kans., St. Louis, Mo., and Chicago, relieving younger officers for duty overseas.

BLACK, DUDLEY FRANK
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 5, 1917. Overseas service Apr. 29, 1918–Apr. 29, 1919. Discharged May 19, 1919. Temporary duty 108th Engrs., Camp Logan; Co. Comdr., 513th Engrs.; with Gen. McKinstry's Special Comm., American Comm. to Negotiate Peace.

BLACK, ERNEST BATEMAN
Entered service Dec. 1, 1917, as Capt., Aviation Sec., Sig. C., N. A.; Maj., A. S., U. S. A., Nov. 5, 1918. Discharged Dec. 16, 1918. Chf. Engr., War Credits Bd.; Engr., Sec. B, Constr. Div.

BLACK, GURDON GILMORE
Entered service May 15, 1917; Capt., E. O. R. C., July 5, 1917; Maj., Engr. R. C., June 12, 1918. Overseas service June 12, 1918–May 26, 1919. Discharged June 19, 1919. Capt. and Adj., 314th Engrs.; Bn. Comdr., 314th Engrs. Toul Sector, St. Mihiel, Euvezio Sector, Meuse-Argonne operations.

BLACK, JAMES BUCKLEY
Entered service May 15, 1917; 1st Lt., E. O. R. C., June 23, 1917; Capt., Engrs., N. A., Aug. 3, 1918. Overseas service Sept. 14, 1917–Jan. 3, 1919. Discharged Jan. 9, 1919. With 34th Engrs.; Office of Chf. Engr., A. E. F.

BLACK, RALPH PETERS
Entered service Jan. 2, 1918; 1st Lt., Engr. R. C., Sept. 28, 1917. Overseas service June 15, 1918–June 9, 1919. Discharged July 3, 1919. With 32d Engrs. on camp constr., St. Sulpice, France.

BLACK, ROGER DERBY
Entered service July 25, 1900; through all grades in C. of E., U. S. A., to Maj., May, 1917; Lt. Col., Aug. 1917; Col., Feb. 1, 1918. Overseas service June, 1917–May, 1918. Resigned July 10, 1919. With 13th Engrs.; C. O., 116th Engrs.; War Plans Div., Gen. Staff. Ordre de l'Etoile Noire.

BLACK, WALTER GLEN
Entered service Sept. 16, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 2, 1918. Co. 7, E. O. T. S., Camp Humphreys.

BLACK, WILLIAM MURRAY
Entered service July 1, 1873; through all grades in C. of E., U. S. A., to Maj. Gen., Chf. of Engrs., Oct., 1917. Overseas service Feb. 22, 1918–Apr. 20, 1918. Retired from active service Oct. 31, 1919. Chf. of Engrs. throughout the war; Chairman, Waterways Comm. of Ry. Administration; Member, Military Committee, Research Council. Distinguished Service Medal.

BLACKMAN, JOHN WILLIAM BERNARD
Entered service Jan. 28, 1917, as Lt., Canadian Ry. Troops: Capt., Engrs., Imperial War Office, Dec., 1918. Overseas service Aug. 11, 1917–Apr. 5, 1919. Discharged Apr. 11, 1919. Special eng. duty; on Chf. Water Engrs. Staff in chg. forty water supplies in France and British Isles.

BLACKWELL, LINUS CRAMBLET
Entered service Dec. 28, 1917; 1st Lt., Engrs., N. A., Mar. 5, 1918. Overseas service July 7, 1918–Apr. 7, 1919. Discharged May 1, 1919. With 602d Engrs., in organization work and as Co. Comdr.; with 5th Army Corps during St. Mihiel and Meuse-Argonne offensives.

BLAIR, ALEXANDER
Entered service June 15, 1916, as Pvt., Inf., Canadian O. T. C.; 2d Lt., Royal Engrs., B. E. F., Mar. 17, 1917; 1st Lt., Royal Engrs., Sept. 17, 1918. Overseas service Oct. 15, 1916–Sept. 25, 1919. Discharged Sept. 27, 1919. With 211th Field Co.; Independent Air Force, B. E. F.; constr. camps and airdromes.

BLAIR, McCREA PARKER
Entered service Mar. 3, 1916, as Capt., Inf., C. E. F.; Maj., Inf., C. E. F., Dec. 3, 1916; Capt., Inf., C. E. F., Apr., 1917. Discharged Mar. 9, 1919. With 221st Bn.; Officer, 1st Class, Depot Bn., Manitoba Regt.

BLANCHARD, MURRAY
Entered service Sept. 2, 1917; Capt., Engrs., N. A., July 20, 1917; Maj., Engrs., N. A., Dec. 11, 1917. Overseas service Aug. 27, 1918–Jan. 5, 1919. Discharged Jan. 8, 1919. With 111th Engrs.; C. O., 520th Engrs. in U. S. and France; miscellaneous construction.

BLAUVELT, LOUIS DAVID
Entered service May 17, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged Aug. 2, 1919. Asst. Const. Q. M., Edgewood Arsenal; Const. Officer, Western Cartridge Co., Alton, Ill.; Const. Officer, Ft. Bliss, Tex.

BLUCHER, CONRAD MEULY von
1st Lt., Engrs., U. S. A.*

BLY, EDWIN PRESCOTT
Pvt., Sig. C., U. S. A.*

BOESCH, CLARENCE EDWIN
Entered service Sept. 13, 1916; Capt., Engrs., N. C. N. G., Aug. 3, 1916; Maj., Engrs., U. S. A., May 12, 1919. Overseas service May 27, 1918–May 31, 1919. Discharged June 21, 1919. Co. Comdr. and Regimental Adj., 105th Engrs.; Asst., G-1, and G-1, 89th Div. Two stars.

BOGGS, FRANK CRANSTOUN
Entered service June, 1894; through all grades in C. of E., U. S. A., to Lt. Col., May 15, 1917; Col., Aug. 5, 1917. Overseas service, June 12, 1918–Jan. 22, 1919. C. O., 315th Engrs.; in chg. engr. depots, A. E. F.; Engr. Purchasing Officer, A. E. F.; Gen. Staff. Officier, Legion d'Honneur.

BOLAND, CHARLES JOSEPH
Entered service Oct. 4, 1917, as 1st Lt., A. S., N. A. Overseas service Nov. 22, 1917–Jan. 3, 1919. Discharged Jan. 8, 1919. Const. aviation camps in Tours, France, and Garden City, N. Y.

BOLGER, EDWIN GIBSON
Pvt., A. S., U. S. A.*

BOLIN, HARRY WILLIAM
Entered service Oct. 12, 1917, as Pvt., Engrs., N. A.; Sgt., Engrs., N. A., Nov. 1, 1917. Overseas service, Mar. 29, 1918–May 10, 1919. Discharged May 15, 1919. With 23d Engrs.; graduate, Engr. School, Langres, France.

BOLTON, FRANK LEONARD
Entered service Sept. 25, 1917; Capt., E. O. R. C., Aug. 4, 1917. Discharged Apr. 2, 1919. With 20th Engrs.; Adj. at Camp Belvoir, Va.; Personnel Adj., Camp Humphreys; Asst. Personnel Adj., Camp Forrest.

BOND, PAUL STANLEY
Entered service June 13, 1900; through all grades in C. of E., U. S. A., to Col., Aug. 5, 1917. Overseas service Jan. 29, 1918–July 18, 1919. Div. Engr., 32d Div., 107th Engrs.; Commandant, Army Engr. and Gas Schools. Two stars.

BONNER, JOHN POLLOCK
Entered service Aug. 28, 1917; Capt., C. A. C., N. A., Nov. 27, 1917. Overseas service Dec. 11, 1917–Oct. 2, 1918. Discharged Dec. 20, 1918. C. O., 101st Trench Mortar Battery; C. O., 2d Trench Mortar Battery; Asst. Director, Trench Artillery, U. S. A. Four stars.

BOORMAN, KITCHELL MONCKTON
Entered service Sept. 15, 1917, as Sapper, Canadian Engrs.; 2d Lt., A. S., N. A., Feb. 7, 1918. Overseas service Mar. 4, 1918–Mar. 17, 1919. Discharged July 11, 1919. Constr.

Officer, Air Service Depot, Romorantin, France, and with 26th Engrs. on water supply at front, 2d Army, A. E. F.

BOOTH, RAYMOND
Entered service Dec. 28, 1917; 1st Lt., Engrs., N. A., Feb. 14, 1918. Overseas service June 30, 1918–July 29, 1919. Discharged Aug. 19, 1919. With 54th Engrs. as Supply Officer; Ry. Transport Officer, Bordeaux, France.

BOOZ, HORACE COREY
Entered service Oct. 3, 1917, as Maj., E. O. R. C.; Col., T. C., N. A., Oct. 29, 1917. Overseas service Oct. 9, 1917–Aug. 18, 1918. Discharged Sept. 14, 1918. Engr. of Constr. on staff Brig. Gen. Atterbury, Director Gen., Transportation, A. E. F., in chg. design of railroad facilities, piers, wharves, storage yards, etc. Officier, Legion d'Honneur; Citation from Gen. Pershing for meritorious service.

BORCHERS, PERRY ELMER
Entered service Aug. 30, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 19, 1918. Camp Humphreys.

BORDEN, GUY
Entered service June 28, 1918, as Pvt., Engrs., N. A.; 2d Lt., Engrs., U. S. A., Oct. 25, 1918. Discharged Jan. 7, 1919. With 165th Depot Brigade; Engr. Replacement Troops.

BOSCHKE, GUY
Entered service July 14, 1917, as Capt., Engrs., N. A.; Maj., Engrs., U. S. A., Aug. 10, 1918. Overseas service July 24, 1917–Jan. 2, 1919. Discharged Jan. 10, 1919. Special duty in dock constr. and incidental installations, A. E. F.; in chg. constr. Bassens docks, Bordeaux, France; investigation and report on ammunition dumps, British front.

BOTT, CLARENCE NICHOLAS
Entered service May 8, 1917; 1st Lt., E. O. R. C., June 13, 1917; Capt., Engr. R. C., May 22, 1918. Overseas service Aug. 26, 1918–June 30, 1919. Discharged Aug. 2, 1919. With 312th and 533d Engrs.

BOULT, CHARLES NORTON
Entered service Feb. 1, 1917, as 2d Lt., Engrs., U. S. A. Overseas service June 10, 1917–Oct. 30, 1918. Discharged Dec. 31, 1918. With 2d Engrs. in Ypres salient and on the Somme.

BOWE, THOMAS FRANCIS
Entered service Aug. 7, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Apr. 1, 1919. Chf. of Engr.'s Staff, Port of Embarkation, Newport News, Va.

BOWEN, CHARLES KENNARD
Entered service Sept. 23, 1918; Capt., Engrs., N. A., Mar. 23, 1918. Discharged Dec. 23, 1918. With 81st Engrs., Ft. Benjamin Harrison, Ind., organization and training of engr. troops.

BOWEN, EDWARD WITHERS
Entered service Aug. 25, 1917; Capt., Inf., U. S. A., Nov. 27, 1917; Maj., Inf., U. S. A., Sept. 14, 1918. Discharged Dec. 19, 1918.

BOWIE, WILLIAM
Entered service Aug. 17, 1918, as Maj., Engrs., U. S. A. Discharged Feb. 28, 1919. Div. of Mapping, Office of Chf. of Engrs.

BOWLBY, HENRY LEE
Entered service Sept. 13, 1917, as Capt., Engrs., N. A.; Maj., Engrs., N. A., Dec. 22, 1917; Lt. Col., Engrs., N. A., July 30, 1918. Overseas service May 22, 1918–Aug. 21, 1918. Discharged Jan. 13, 1919. Regtl. Adj. with 20th Engrs.; organization work with 41st, 42d and 43d Engrs.; C. O., 43d Engrs.; Field Officers' School, Langres, France; Dir., Emergency Bridge School, Camp Humphreys. One star.

BOWLES, JAMES TEN BROECK
Entered service Aug. 5, 1917, as Maj., San. C., N. A.; Lt. Col., San. C., Mar. 9, 1918. Overseas service Mar. 12, 1918–June 27, 1918. Discharged June 27, 1918. Surgeon Gen.'s Office, Water Supply Section, Washington, D. C.; with Chf. Engr., A. E. F., water supply service.

BOWNE, SIDNEY BREESE
Entered service Mar. 25, 1918, as 2d Lt., Public Health Service; 1st Lt., Public Health Service, Mar. 15, 1919. Released from active duty Apr. 1, 1919. San. Engr., Park Field, Tenn.

BOYD, GEORGE RAY
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 28, 1917. Overseas service Jan. 20, 1918–Feb. 22, 1919. Discharged Feb. 28, 1919. With 303d Engrs., Camp Dix; 116th Engrs., A. E. F.

BOYDEN, HARRY CHESTER
Entered service July 21, 1917; Capt., E. O. R. C., June 23, 1917; Maj., Engrs., N. A., Apr. 4, 1918. Discharged Oct. 31, 1919. Asst. to Dept. Engr., in chg. collection and compilation military information for progressive military map in Western Dept.; in chg. for whole U. S. of same work; Asst. and Acting Dept. Engr., Central Dept.; Instr. at Engr. School, Camp Humphreys.

BOYLE, JAMES MURRAY
Entered service Jan., 1918, as Maj., Ord. Eng., U. S. A. Released from active service Dec. 31, 1919. Originating, designing and superv. constr. and test of spherographic system, anti-aircraft fire control.

BRACE, JAMES HENRY
Entered service Sept. 11, 1918, as Maj., Engrs., N. A. Overseas service Oct. 4, 1918–Feb. 26, 1919. Discharged Feb. 28, 1919. With 22d Engrs. in light ry. work at front, attached to 12th Engrs. with 2d Army, A. E. F. Citation from Gen. Pershing for meritorious service.

BRAGG, KENDALL BENJAMIN
Entered service June 15, 1917, as Lt., Jr. Grade, C. E. C., U. S. N.; Lt., C. E. C., U. S. N., Oct. 15, 1917. Naval Academy; Naval Air Station, Pensacola, Fla.; Bureau of Yards and Docks, Navy Dept., Washington, D. C.

BRALY, RAYMOND FIELDING
Entered service May 5, 1917, as 1st Lt., E. O. R. C. Discharged May 1, 1919. Engr. Depot Officer, Warehousing Officer, Camp Supply Office, Camp Shelby, Miss.

BRAUNWORTH, PERCY LEWIS
Entered service Sept. 1, 1917; Capt., E. O. R. C., June 23, 1917. Overseas service Oct. 28, 1917–Jan. 24, 1919. Discharged Jan. 28, 1919. Attached to Div. of Light Rys. and Roads, A. E. F.; Asst. Supt., and Supt. of Roads, Base Sec. No. 2, A. E. F.; with 135th Engrs.

BRECK, CHARLES RENWICK, JR.
Entered service Jan. 31, 1918, as Capt., T. C., N. A. Overseas service June 6, 1918–June 11, 1919. Discharged July 1, 1919. Special duty with T. C. at La Rochelle, Les Sables d'Olonne, Tours, and Le Mans, France.

BRES, EDWARD SEDLEY
Entered service May 15, 1917; 1st Lt., E. O. R. C., June 21, 1917; Capt., Engrs., N. A., July 31, 1918; Maj., Engrs., U. S. A., Aug. 9, 1919. Overseas service Aug. 22, 1918–Apr. 25, 1919. Discharged June 6, 1919. With 114th Engrs.; 1st Army Corps, A. E. F.

BREUCHAUD, JULES ROWLEY
Entered service Oct. 25, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Dec. 13, 1918.

BREWSTER, HENRY BAUM
Entered service May 12, 1917; Capt., E. O. R. C., June 16, 1917; Maj., Engrs., U. S. A., May 2, 1919. Overseas service June 24, 1918–June 9, 1919. Discharged June 12, 1919. Adj., 303d Engrs.; assisted in organization of 66th Engrs.; with 66th Engrs.; Peace Comm. in assessment of war damages in Roumania; special duties, Office, Chf. of Engrs. Citation from Gen. Pershing for meritorious and conspicuous service.

BREYMANN, JOHN BERNARD, JR.
Entered service Dec. 19, 1917, as Pvt., C. A. C., N. A.; 2d Lt., C. A. C., N. A., Mar. 27, 1918. Discharged Jan. 7, 1919. Coast defenses, Puget Sound; 39th Heavy Artillery, Camps Upton and Grant.

BRIGGS, ROBERT WESLEY
Entered service Dec. 3, 1917; 2d Lt., Aviation Sec., Constr. Div., N. A., Nov. 18, 1917; 1st Lt., Engrs., N. A., Oct. 18, 1918. Overseas service Sept. 29, 1918–Mar. 7, 1919. Discharged Mar. 15, 1919. Constr., and training troops while in U. S.; with 467th Engrs. in France.

BRIGHT, GRAHAM BERNARD
Entered service May 8, 1917; Capt., Engrs., N. A., Aug. 15, 1917; Maj., Engrs., N. A., Feb. 7, 1918. Overseas service May 24, 1918–Sept. 3, 1919. Discharged Oct. 11, 1919. With 305th Engrs.; with 6th Engrs. in Army of Occupation; Graduate of Gen. Staff College, France. Three stars.

BRINKLEY, MILO HAMILTON
Entered service June 26, 1918, as Capt., Engrs., N. A. Discharged Sept. 30, 1919. Historical Data Section, Office, Chf. of Engrs.; Co. Comdr., 138th Engrs.

BROCKMAN, GEORGE FRED
Entered service Oct. 1, 1917, as 1st Lt., Engrs., N. A. With U. S. G. S., military mapping, Southeast States and training camps.

BRONSON, HOWARD FRANKLIN
Entered service Dec. 9, 1917; Capt., San C., N. A., Nov. 27, 1917. Discharged Aug. 14, 1919. Camp San. Engr., Camp Upton.

BROOK, ALEC EDWARD
Entered service Sept. 14, 1914, as Pvt., Inf., British Army; 2d Lt., Royal Engrs., Aug. 19, 1915; 1st Lt., Royal Engrs., June 1, 1916; Capt., Royal Engrs., Feb. 27, 1918; Maj. (temporary) Royal Engrs., June 19, 1918. Overseas service from enlistment to discharge (resides in England). Discharged Feb. 22, 1919. With 12th Bn. Inf., York and Lancaster Regt.; Territorial Force, Royal Engrs.; with light ry. units at front. Seven stars.

BROOKING, JOSEPH HUGH
Entered service June 18, 1917, as 1st Lt., E. O. R. C.; Capt., E. O. R. C., Mar. 30, 1918. Overseas service July 28, 1917–Apr. 27, 1919. Discharged May 16, 1919. Co. Comdr., 12th Engrs.; commanded light ry. unit attached to British Army; light ry. constr. in Vosges, and in chg. at St. Mihiel; with 2d Army, A. E. F., on Toul front. Nine stars.

BROOKS, JOSIAH RICHARDSON
Entered service Dec. 28, 1917; 1st Lt., E. O. R. C., Aug. 30, 1917; Capt., Engrs., U. S. A., May 30, 1919. Overseas service June 29, 1918–July 14, 1919. Discharged July 23, 1919. With 28th and 130th Engrs.

BROWER, IRVING CLINTON
Entered service May 18, 1918, as Capt., Q. M. C., Constr. Div., N. A.; Maj., Q. M. C., Constr. Div., U. S. A., Nov. 1, 1918. Discharged May 15, 1920. Executive Officer, Utilities Dept.; in chg. wharves, roads, railroads, and drainage.

BROWN, ALFRED THOMAS
Entered service June, 1917, as Lt., E. O. R. C. Overseas service ———— to Jan. 22, 1919. Discharged Mar. 7, 1919. With Gen. Staff; with 1st and 3d Armies, A. E. F. Three stars. One wound.

BROWN, CHARLES EUGENE
Capt., Engrs., U. S. A.*

BROWN, CLARENCE COWGILL
Entered service May 8, 1917; 1st Lt., E. O. R. C., June 18, 1917; Capt., Engrs., U. S. A., Sept. 20, 1918. Overseas service Aug. 14, 1917–May 14, 1919. Discharged June 2, 1919. With 12th Engrs. on light ry. work; Picardy front with British Forces one year, remainder of service with American Forces in Lorraine.

BROWN, EARL IVAN
Entered service June 14, 1894; through all grades in C. of E., U. S. A., to Lt. Col. May 17, 1917; Col. (temporary), Aug. 20, 1917. Overseas service June 9, 1917–June 8, 1918. Organized 307th Engrs.; organized and commanded 317th Engrs. in overseas service; Chf. Engr., 5th Army Corps. Two stars.

BROWN, ELBERT CALVIN
Entered service June 17, 1918, as Capt., Engrs., N. A. Discharged Apr. 5, 1919. With 217th Engrs.

BROWN, ELLIOT CHIPMAN
Entered service May, 1917, as Lt., U. S. N. R. F.; Lt. Comdr. C. E. C., U. S. N., Sept., 1917; Comdr., June, 1919. Overseas service July 10, 1918–Sept. 20, 1918. Released from active duty Dec. 23, 1918. In chg. constr., 3d Naval Dist. outside Brooklyn Navy Yard; Public Works Officer, in chg. constr. Hampton Roads Naval Operating Base; Constr. Mgr., Pelham Bay Naval Camp; Aide for eng., Asst. Secy. of the Navy, in chg. constr., Newport Training Station.

BROWN, HORACE TROWBRIDGE
Entered service Apr. 9, 1918, as Pvt., Engrs., N. A.; Cpl., Engrs., N. A., July 9, 1918; Sgt., Engrs., U. S. A., Oct. 6, 1918; 2d Lt., Engrs., U. S. A., May 14, 1919. Overseas service May 21, 1918–July 13, 1919. Discharged July 19, 1919. With 43d and 20th Engrs.; St. Aignan Replacement Camp; constr. light rys., Dept. of Cote d'Or, France.

BROWN, HORATIO WHITTEMORE
Entered service May 7, 1917; 2d Lt., E. O. R. C., Aug. 15, 1917; 1st Lt., C. W. S., Nov. 13, 1918. Overseas service Dec. 11, 1917–July 29, 1919. Discharged Aug. 15, 1919. Asst., and Acting Director, Army Gas School, Langres, France; Asst. Div. Gas Officer, 26th Div.

BROWN, LEO FRANCIS
Entered service May 26, 1918, as Pvt., Inf., N. A.; Sgt., Inf., N. A., July 12, 1918. Discharged Jan. 6, 1919. With Billeting Officer, Camp Upton; Candidate, O. T. S., Camp Gordon.

BROWN, LEVANT R.
Entered service June 26, 1918; Capt., E. O. R. C., May 16, 1917. Discharged Jan. 14, 1919. Co. Comdr., 2d Engr. Training Regt., Camp Humphreys; Adj., 99th Engrs.; Adj., 76th Engrs., Camp Leach.

BROWN, MARSHALL WRIGHT
Entered service June 26, 1917; Maj., E. O. R. C., May 7, 1917; Lt. Col., Engrs., U. S. A., Apr. 7, 1919. Overseas service June 27, 1917–Oct. 1, 1919. Discharged Oct. 28, 1919. With Line of Communications, A. E. F.; with Engr. Purchasing Officer, A. E. F.: with Sec. Engr., Advance Sec., A. E. F., G. S. A., A. E. F. Legion d'Honneur (French); St. Sava (Serbia).

BROWN, NORMAN FREED
Entered service Jan. 24, 1918, as Maj., T. C., N. A. Overseas service Feb. 7, 1918–Mar. 29, 1919. Discharged Apr. 1, 1919. Asst. to Engr. of Constr., staff of Director Gen. Transportation, A. E. F.; Member of Inter-Allied Comm. for reception of German ry. material. Citation from Gen. Pershing for meritorious service.

BROWN, ROBERT HUSE
Entered service Sept. 19, 1917, as Capt., San. C., N. A.; Maj., San. C., U. S. A., Aug. 3, 1918. Discharged Dec. 9, 1918. Gen. san. work, Camp Greenleaf; San. Insp. and San. Engr., Camp Humphreys; office, Surgeon Gen. of the Army.

BROWNELL, ERNEST HENRY
Entered service Oct. 24, 1902; successive promotions to Comdr., C. E. C., U. S. N., July 1, 1917. Overseas service Nov. 3, 1917–Mar. 23, 1918. Bureau of Yards and Docks, Navy Yard, Portsmouth, N. H.; established overseas shore stations for sea-planes and dirigibles, also fuel stations.

BRUNER, LOUIS SCHUMANN
Entered service May 12, 1917; 1st Lt., E. O. R. C., Aug. 15, 1917; Capt., Engrs., N. A., May 9, 1918. Overseas service Mar. 29, 1918–June 8, 1919. Discharged June 19, 1919. With 23d Engrs. in operation of quarries, constr. of roads and maintenance in Base Sec. No. 1, and in Toul Sector with 1st and 2d Armies, A. E. F. Two stars.

BRUSH, CARL FLETCHER
Capt., Engrs., U. S. A., A. E. F.*

BRUTON, PHILIP GILSTRAP
Entered service Aug. 25, 1917; 2d Lt., Sig. C., Constr. Div., N. A., Nov. 8, 1917. C. O., 26th Constr. Squadron on ry. constr. in Northwest.

BRYAN, CLARK ALBERT
Entered service Nov. 28, 1917, as Capt., Engrs., N. A. Discharged Dec. 28, 1919. Const. Q. M. at Camp Beauregard and Ft. Benjamin Harrison, Ind.

BRYAN, GEORGE, JR.
Entered service Oct. 9, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 11, 1918.

BUCHANAN, WARREN HENDERSON
Entered service July 10, 1918, as Chf. Machinist's Mate, U. S. N. R. F.; Ensign, U. S. N. R. F., Dec. 31, 1918; Ensign, U. S. N., Apr. 22, 1919. Discharged Mar. 20, 1920. Public Works Officer, Naval Training Station, Gulfport, Miss., and Aide to Commandant; C. O. 3d Regt.

BUCHER, HAROLD FOLLMER
Entered service May 9, 1917; 2d Lt., E. O. R. C., June 13, 1917; 1st Lt., Engr. R. C., June 7, 1918; Capt., Engrs., U. S. A., May 28, 1919. Overseas service Sept., 1917–July, 1918, and Sept., 1918–July, 1919. With 2d Engrs. at Toul, Verdun and Chateau Thierry; Meuse-Argonne offensive. Four stars.

BUCK, RICHARD SUTTON
Entered service June 7, 1917, as Maj., E. O. R. C. Overseas service July 14, 1917–Dec. 22, 1918. Discharged Jan. 7, 1919. With 11th Engrs.; Staff duty 4th Bn., Canadian Ry. Troops; Staff duty, Base Sec. No. 2, A. E. F. Distinguished Service Order, Great Britain.

BUCK, WALTER VAN
Entered service May 13, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service Mar. 30, 1918–June 16, 1919. Discharged July 7, 1919. With 23d Engrs.; Co. Comdr., ry., building, and road constr. work at Chaumont, Neufchateau, Nevers, France; with 1st Army, A. E. F., in Meuse-Argonne offensive; highway reconstr., Advance Sec., S. O. S., A. E. F. One star.

BUCKWALTER, HARRIS DANIEL
Entered service Sept. 5, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service, Jan. 27, 1918–May 11, 1918. Camp American University; special duty with 307th Engrs., Camp Gordon; Co. Comdr. and Adj., 1st Engrs. Killed May 11, 1918, in action at Belle Assise Farm, France.

BULL, GEORGE MAIRS
Lt. Comdr., C. E. C., U. S. N.*

BUNKER, GEORGE HITCHELL
Entered service May 8, 1917; Capt., E. O. R. C., Aug., 1917; Maj., Engrs., N. A., May 21, 1918; Lt. Col., Engrs., U. S. A., Mar. 21, 1919. Overseas service July 31, 1918–June 15, 1919. Discharged July 9, 1919. With 306th Engrs.; training engr. troops. Divisional citation, 81st Div. One star.

BUNKER, STEPHEN SANS
Entered service May 15, 1917; Capt., E. O. R. C., July 13, 1917. Overseas service Nov. 26, 1917–July 17, 1919. Discharged Aug. 3, 1919. With 503d and 72d Engrs., lumber production; storage depot constr., Base Sec. No. 1, A. E. F.

BURD, EDWARD MORRIS
Entered service Aug. 27, 1917; Capt., C. A. R. C., Nov. 27, 1917. Discharged Apr. 8, 1919. Instr., Coast Arty. School, Fort Monroe, Va.

BURDETT, OWEN LONG
Entered service May 8, 1917; Capt., E. O. R. C., Sept., 1917. Overseas service Oct. 31, 1917–Mar. 24, 1919. Discharged Mar. 31, 1919. With Sec. Engr., Base Sec. No. 2, A. E. F.; camp constr. with 1st Army; Co. Comdr. 25th Engrs., gen. constr., Meuse-Argonne offensive. One star.

BURGER, ALFRED ANDREW
Entered service Oct. 4, 1917, as Pvt., Inf., Depot Brig.; Cpl., Oct. 16, 1917; Sgt., Apr. 29, 1918; 2d Lt., Inf., U. S. A., June 1, 1918. Discharged Dec. 21, 1918. Depot Brig., 83d Div.; Replacement Regt., Camp Gordon; Machine Gun Training Center, Camp Hancock.

BURGESS, HARRY
Entered service June 17, 1891; through all grades in C. of E., U. S. A., to Col., July 6, 1917. Overseas service Aug. 1, 1917–Apr. 22, 1919. C. O., 16th Engrs.; Chf. Engr., 5th Army Corps; C. O., 305th Engrs. and Chf. Engr., 80th Div. Companion of the Order of St. Michael and St. George, Great Britain. Four stars.

BURKETT, JOSEPH MILLER †
Entered service Aug. 25, 1917; Capt., Engrs., N. A., July 25, 1917. Overseas service Jan. 26, 1918–July 29, 1919. Discharged Aug. 29, 1919. Vancouver Barracks, Wash., training troops; with 116th Engrs. in France; with Engr. Purchasing Officer, Paris; Insp. Gen. Dept., Bordeaux, France.

† Died April 14, 1921.

BURKY, CHARLES ROGY
Entered service Aug. 25, 1917; Capt., Sig. R. C., A. S., N. A., Nov. 8, 1917. Discharged Jan. 10, 1919. With 42d Spruce Squadron near Aberdeen, Wash.

BURR, EDWARD
Entered service June 12, 1878; through all grades in C. of E., U. S. A., to Brig. Gen., Aug. 5, 1917. Overseas service July 14, 1918–Feb. 17, 1919. Permanent rank assumed Feb. 5, 1919. In command 4th Engrs.; in command 166th F. A. Brig.; in command 62d F. A. Brig. Two stars.

BURRELL, GLENN SMITH
Entered service July 30, 1907; through all grades in C. E. C., U. S. N., to Lt. Comdr., July 1, 1917; Comdr., Mar., 1919. Public Works Officer, Naval Station, Guam; Bureau of Yards and Docks, Washington, D. C.; Public Works Officer in chg. constr. submarine base and ammunition depot at New London, Conn.

BURROWES, HARRY GILBERT
Entered service May 12, 1917; 1st Lt., E. O. R. C., June 23, 1917; Capt., Engrs., U. S. A., Oct. 17, 1918. Overseas service Oct. 31, 1917–Apr. 12, 1919. Discharged July 9, 1919. With 25th Engrs.; on road constr., St. Nazaire; hosp. constr., Angers; various duties at Is-sur-Tille almost entirely on detached service; 1st Army, A. E. F., Meuse-Argonne offensive. One star.

BURT, HENRY JACKSON
Entered service Feb. 20, 1918, as Maj., Q. M. C., N. A. Discharged May 2, 1919. Expediting Officer, Eng. Branch, Washington, D. C.; on Bd. of Contract Review, Bd. of Sales, and Special Bd. to Investigate Constr. Barges at Savannah, Ga.

BURTON, WILLIAM
Entered service May 8, 1917; 1st Lt., E. O. R. C., Jan. 15, 1917; Capt., Engrs., N. A., Apr. 9, 1918. Overseas service Aug. 29, 1918–June 23, 1919. Discharged July 17, 1919. In office Director Gen. of Military Railways, Washington, D. C. Co. Comdr., 521st Engrs.

BUSH, LEE MARSHALL
Entered service June 6, 1918, as Pvt., Cavalry, U. S. A.; Sgt., F. A. Replacement troops, July 18, 1918; 2d Lt., F. A., Oct. 23, 1918. Discharged Dec. 8, 1918. With Battery B, 10th Regt.; Instr. in reconnaissance, Camp Jackson.

BUSH, LINCOLN
Entered service Jan. 3, 1918, as Maj., Q. M. C., N. A.; Lt. Col., Q. M. C., Constr. Div., N. A., Mar. 18, 1918; Col., Q. M. C., Constr. Div., U. S. A., Aug. 24, 1918. Discharged Mar. 31, 1919. Assoc. Officer in chg. eng. branch, Constr. Div., on work in U. S., new port terminals and associated projects.

BUSHNELL, HOWARD BLAINE
Entered service Sept. 1, 1917, as Capt., Engrs., N. A. Overseas service, Jan. 23, 1918–Feb. 17, 1919. Discharged Feb. 19, 1919. Chf. constr. Officer, Aerial Gunnery School and Ord. Schools, St. Jean des Mont, France; Asst. Supt. Roads and Quarries, Base Sec. No. 1, A. E. F.

BUSHNELL, HOWARD EMORY
Entered service Aug. 28, 1917, as Capt., Ord. Dept., N. A.; transferred to Engrs., N. A., May 27, 1918. Discharged Jan. 6, 1919. In chg. constr. Picatinny Arsenal, Dover, N. J.; Personnel Adj., Camp Humphreys; Adj., 217th Engrs., Camp Beauregard.

BUSSE, FRANZ AUGUST
Entered service Sept. 2, 1917; Capt., E. O. R. C., May 16, 1917. Overseas service Jan. 27, 1918–Aug. 8, 1919. Discharged Sept. 3, 1919. Constr. of Intermediate Storage Depot No. 1; Chf. Engr. Office, A. E. F., Div. of Military Eng. and Engr. Supplies.

BUTLER, JOHN SOULE
Entered service Aug. 1, 1917; Maj., E. O. R. C., July 5, 1917; Lt. Col., Engrs., N. A., July 18, 1918. Discharged Aug. 28, 1919. Gen. Engr. Depot, Washington, D. C., in chg. Depot Dept.; in office Director of Finance.

BUTLER, MILLARD ANGLE
Entered service June 6, 1917, as Maj., Q. M. C., N. A.; Col., Q. M. C., Constr. Div., N. A., Mar. 18, 1918. Discharged Mar. 18, 1920. Const. Officer, Camp Dodge, Ft. Dodge Hosp., Norfolk Army Supply Base, Donner Union Coke Corp. plant, Buffalo; Member, Bd. of Control, War Constr. Activities, Hampton Roads Dist.

BUTLER, WILLIAM PARKER
Entered service Aug. 24, 1917, as Ensign, U. S. N. R. F.; Lt. Junior Grade, U. S. N. R. F., Feb. 10, 1918; Lt., U. S. N. R. F., July 15, 1918. Released from active duty, Mar. 15, 1919. Watch officer on duty in naval communications under Chf. of Naval Operations.

BUTTERS, JOHN HENRY
Entered service July 31, 1914; Capt., Engrs., Australian Army, Sept. 1909; Maj., Engrs., Australian Army, Jan. 1918. Released from active duty, Aug., 1920. C. O., Hobart defenses. Honorary Major for meritorious services. Awarded M. B. E.

BYAM, LEROY HENRY
Entered service Apr. 10, 1918; Capt., Engrs., N. A., Apr. 16, 1918; Maj., Engrs., N. A., Oct. 31, 1918. Overseas service, June 30, 1918–Apr. 29, 1919. Discharged May 17, 1919. Engr. Officer in chg. constr. 3d Aviation Instr. Center, Issoudun, France; Div. Engr., Road Repairs, Dept. of Cher, France; with 55th 128th Engrs.

BYLLESBY, HENRY MARISON
Lt. Col., A. S., U. S. A., A. E. F.*

BYRD, JOHN HENRY
Entered service May 12, 1917, as Capt., E. O. R. C. Overseas service May, 1918–June, 1919. Discharged June 20, 1919. Co. Comdr., 314th Engrs. in St. Mihiel and Meuse-Argonne offensives and Toul and Lucy Sectors. Four stars.

BYRNE, THOMAS SNEED
Entered service Sept. 7, 1917; 1st Lt., Ord. Dept., Nitrate Div., July 16, 1917; Capt., Ord. Dept., Nitrate Div., Jan. 21, 1918. Discharged Feb. 16, 1919. Insp. of Constr., U. S. Nitrate Plant No. 1, Sheffield, Ala.; Asst. Dir. Operations, U. S. Nitrate Plant No. 3, Toledo, Ohio.

CAFFALL, GEOFFREY ARTHUR
Entered service July 26, 1917; Sapper, Canadian Engrs., Aug. 8, 1917; Pvt., Canadian Machine Gun Depot, Jan. 15, 1918. Overseas service Mar. 4, 1918–May 25, 1919. Discharged May 25, 1919. O. T. C., Toronto; 2d Bn., 2d Div. Canadian Machine Gun Corps, B. E. F., at Arras, Cambrai, Denain, Valenciennes, and Mons; with British Army of Occupation.

CALDWELL, WILLIAM HOWELL
Entered service Jan. 25, 1918, as Lt., U. S. N. R. F.; Lt. Comdr., U. S. N. R. F., Mar. 29, 1919. Gen. constr. work at proving grounds, Dahlgren, Va.

CAMERON, HARRY FRANK
Entered service May 8, 1917, as Capt., E. O. R. C.; Maj., E. O. R. C., Aug. 15, 1917; Lt. Col., Engrs., U. S. A., Aug. 29, 1918. With 301st Engrs., Camp Devens; Engr. Instr., Camp Lee; C. O., Camp La Grange, Va.; C. O., 4th Engr. Training Regt.; C. O., 209th Engr.; Chf. of Machinery and Engr. Material Div., Office, Director of Purchase, Washington, D. C.

CAMP, GEORGE DASHIELL
Entered service May 1, 1917; 2d Lt., E. O. R. C., Jan. 30, 1917; 1st Lt., E. O. R. C., Aug. 15, 1917. Overseas service June 28, 1918–Apr. 16, 1919. Discharged May 12, 1919. Engr. Instr.; with 316th Engrs.

CAMPBELL, CHARLES CECIL
Entered service June 26, 1918, as Pvt., Medical Corps; Pvt., 1st Class, Medical Corps, July 31, 1918; Cpl., Medical Corps, Nov. 12, 1918. Discharged Feb. 13, 1919. With 15th Bn., Camp Greenleaf; Motor Co. No. 13, Chickamauga Park; Replacement Unit No. 74, Camp Greenleaf; Camp Merritt.

CAMPBELL, GEORGE RAYMOND
Entered service June 11, 1917, as Capt., E. O. R. C. Discharged Dec. 11, 1917.

CANFIELD, GEORGE HATHAWAY
Entered service May 16, 1917, as Capt., E. O. R. C.; Maj., Engr. R. C., May 8, 1918; Lt. Col., Engrs., U. S. A., Sept. 3, 1918. Overseas service June 28, 1918–May 30, 1919. Discharged July 8, 1919. With 316th Engrs., during Argonne and Lys operations; Div. Engr., 91st Div. Three stars.

CAREY, MATTHEW LAURENCE
Entered service Aug. 1, 1917; Capt., Q. M. C., N. A., May, 1917. Overseas service Apr. 1918–Feb., 1919. Discharged Feb., 1919. Transportation Officer, Camp Dix; Asst. to Chf. Q. M., Base Sec. No. 3, A. E. F.; Property Officer and Asst. to Camp Q. M., Liverpool, England.

CAREY, WILLIAM NELSON
Entered service May 8, 1917, as Capt., E. O. R. C.; Maj., Engr. R. C., Mar., 1918. Overseas service July, 1918–June, 1919. Discharged June 7, 1919. Adj., 313th Engrs.; Asst. Chf. Engr., 7th Army Corps, Army of Occupation; Bn. Comdr., 307th Engrs. Three stars.

CARLIN, JOSEPH PATRICK
Entered service Oct. 15, 1917; Capt., Ord. R. C., Nov. 19, 1917. Discharged Jan. 4, 1919. In chg. amortization of plants, including plants commandeered by Ord. Office, Washington, D. C.

CARLSON, CARL ALEXIUS
Entered service June 27, 1903; through all grades in C. E. C., U. S. N., to Comdr., Feb. 1, 1918. Public Works Officer, 12th Naval Dist.; on constr. Pearl Harbor dry dock.

CARO, PHILLIP
Lt., British Army.*

CARR, DEAN ORRIS
Entered service Aug. 10, 1918; Capt., Engr. R. C., Aug. 1, 1918. Discharged Dec. 20, 1918. Gen. constr. work.

CARR, GEORGE WALLIS, JR.
Supply Sgt., Inf., U. S. A.*

CARROLL, JAMES EDWARD
Entered service July 11, 1917, as Maj., E. O. R. C. Overseas service June 22, 1918–June 30, 1919. Discharged July 11, 1919. With 531st Engrs.; C. O. 527th Engrs.; with 1st Army in St. Mihiel and Meuse-Argonne offensives; in chg. of road work (highways), in part of the 1st Army Engr. area. Two stars.

CARTER, EVLANE KEMPER
Entered service Dec. 28, 1917; Capt., Engrs., N. A., Oct. 1, 1917. Overseas service Sept. 3, 1918–June 11, 1919. Discharged July 5, 1919. With 518th Engrs. as Co. Comdr. in constr. work, Base Sec. No. 4 and No. 6, A. E. F.

CASE, CHARLES ANDREW
Entered service May 8, 1917; 2d Lt., E. O. R. C., Feb. 19, 1917; 1st Lt., E. O. R. C., Aug. 15, 1917; Capt., Engrs., U. S. A., Sept. 20, 1918. Overseas service June 12, 1918–May 26, 1919. Discharged June 20, 1919. With 314th Engrs. Citation from Gen. Pershing for distinguished gallantry; Divisional citation 89th Div. Two stars.

CATE, CHARLES EDWARD
Entered service Feb. 25, 1918, as 1st Lt., E. O. R. C.; Capt., Engrs., U. S. A., Apr. 6, 1919. Overseas service July 10, 1918–Aug. 18, 1919. Discharged Aug. 26, 1919. Military ry. constr., Camp Humphreys; Engr., M. of W., 14th Grand Div., T. C., Nantes, France.

CATON, JOHN HIRST
Entered service Sept., 1918; Capt., Engr. R. C., July 10, 1918; Maj., Engrs., U. S. A., Apr. 11, 1919. Overseas service June 29, 1918–July 25, 1919. Discharged Aug. 4, 1919. Co. Comdr., and Bn. Comdr., 33d Engrs.; Road Engr., Base Sec. No. 5, A. E. F.

CATTELL, WILLIAM ASHBURNER†
Entered service Dec. 28, 1917; Maj., E. O. R. C., Mar. 1, 1917. Discharged Sept. 30, 1919. On staff duty in Office, Chf. of Engrs., in chg. Historical Sec.

CERNY, JOHN WILLIAM
Entered service Feb. 14, 1918, as Maj., Q. M. C., N. A. Discharged Sept. 25, 1919. Superv. Const. Q. M., Washington, D. C.; Asst. Const. Q. M., Brooklyn Army Supply Base.

CHADWICK, MAURICE FOSTER
Cpl., F. A., U. S. A.*

CHAMBERLAINE, ROBERT LLOYD
Entered service Jan. 30, 1918, as Capt., A. S., N. A. Discharged Dec. 23, 1918. In chg. Programme and Statistics Dept., Bureau of Aircraft Production.

CHAMBERS, FRANK TAYLOR
Entered service July 19, 1897; through all grades in C. E. C., U. S. N., to Capt., June, 1914. Overseas service, July 31, 1918–Sept. 30, 1918. Bureau of Yards and Docks, Hampton Roads, Va.; member board to select naval fuel oil base, Chesapeake Bay; member Comm. on Navy Yards and Naval Stations; Engr. in chg. dry docks, Emergency Fleet Corp.; Chf. Engr. and member, Port Facilities Comm., U. S. Shipping Board.

CHANDLER, JOHN HENRY
Entered service Oct. 29, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 23, 1918. Camp Humphreys.

CHANDLER, WILLIAM EUSTACE
Entered service Aug. 20, 1917, as Pvt., C. A. C.; Master Gunner, C. A. C., Dec. 21, 1917; 2d Lt., C. A. C. Overseas service Mar. 25, 1918–Jan. 22, 1919. Discharged Feb. 12, 1919. Boston Harbor Coast Defenses; with 55th Regt., C. A. C.; graduated from Heavy Artillery School, France; Vesle River Sector; 2d Battle of Marne; Meuse-Argonne offensive. One star.

CHAPMAN, MELVILLE DOUGLAS
Entered service May 26, 1917, as Capt., U. S. M. C. Overseas service 1918–1919. Released from active duty Sept., 1919. Brooklyn Navy Yard; on Inter-Allied Economic Comm.: Economic and Finance Officer, Treves and Coblenz, Germany; Adj., 5th Brig. Machine Gun Bn.

CHAPMAN, STANLEY ALBA
Entered service Dec. 28, 1917; 1st Lt., E. O. R. C., Aug. 1, 1917. Overseas service Apr. 10, 1918–July 5, 1919. Discharged July 10, 1920. Special duty with French transportation eng. material; Instr. in Road Engineering, A. E. F. Univ., Beaune, France.

CHAPPELL, CLAUDE EDWARD
Entered service Aug. 28, 1918; Capt., Engrs., U. S. A., Sept. 7, 1918. Discharged Jan. 11, 1919. Camp Humphreys.

CHARNLEY, WALTER
Entered service July 29, 1918, as 1st Lt., Engrs., N. A. Discharged Jan. 25, 1919. With 70th Engrs.

CHASE, CLIFFORD EARL
Entered service May, 1917; 1st Lt., Engrs., N. A., Aug., 1917; Capt., Engrs., N. A., Feb., 1918; Maj., Engrs., U. S. A., Oct., 1918. Overseas service Apr. 1918–Dec. 24, 1918. Discharged June 5, 1919. With 4th Engrs. Four stars. Two wounds.

CHASE, DEAN‡
Entered service Aug., 1917; 1st Lt., C. A. R. C., Nov. 27, 1917. Overseas service, July 13, 1918–Feb. 19, 1919. Discharged Mar. 8, 1919. Regtl. Orienteur, 62d Regt., C. A. C., and Staff Officer.

CHENEY, SHERWOOD ALFRED
Entered service June 15, 1893; through all grades in C. of E., U. S. A., to Lt. Col., May 15, 1917; Col., Aug. 7, 1917; Brig. Gen., Oct. 11, 1918. Overseas service June 2, 1917–July 23, 1917, and May 2, 1918–Feb. 23, 1920. Permanent rank taken Oct. 31, 1919. In command 110th Engrs.; Asst. to Chf. Engr., A. E. F., at G. H. Q.; Director, A. T. S. Distinguished Service Medal; Commander, Legion d'Honneur. One star.

CHERRY, ALAN GORDON
Entered service May 11, 1917; Pvt., Engrs., N. A., Sept. 1, 1917; 2d Lt. Engrs., N. A., Jan. 15, 1918; 1st Lt., Engrs., N. A., June 15, 1918. Overseas service July 14, 1918–June 13,

† Died Oct. 17, 1920.

‡ Died May 26, 1919.

1919. Discharged July 7, 1919. With 301st Engrs.; special duty Operations Sec., Chf. Engr. Office, 3d Army; St. Mihiel offensive; Army of Occupation. Two stars.

CHEVALIER, WILLARD TOWNSHEND
Entered service May 31, 1917; Capt., E. O. R. C., June 13, 1917; Maj., Engrs., U. S. A., Aug. 10, 1918; Lt. Col., Engrs., U. S. A., Feb. 26, 1919. Overseas service July 14, 1917–Apr. 27, 1919. Discharged May 5, 1919. Co. Comdr., Regtl. Adj., Bn. Comdr., and Regtl. Comdr., 11th Engrs. Citation from Gen. Pershing for meritorious and conspicuous service at Vraincourt, France. Four stars.

CHILD, JOHN TOWNSHEND
Entered service May 25, 1918, as Pvt., Inf., N. A.; 2d Lt. Engr. Sec., San. C., N. A., June 13, 1918; 1st Lt., Engr. Sec., San. C., U. S. A., Nov. 9, 1918. Discharged Jan. 25, 1919. Co. Comdr., Camp Meade, on drainage work, mosquito control and gen. camp sanitation.

CHILDS, JAMES ALANSON
Entered service Sept. 20, 1918, as Capt., San. C., N. A. Discharged Mar. 12, 1919. Camp San. Engr., Camp Jackson.

CHIPLEY, DUDLEY
Entered service July 25, 1918, as Capt. Q. M. C., Constr. Div., N. A. Discharged Mar. 25, 1919. Officer in chg. water and sewers at Camp Hancock; Utility Officer, Camp Benning; Port Eng. Utilities Office, Newport News, Va.

CHITTENDEN, ALBERT FREDERICK
Entered service June 15, 1917, as 1st Lt., E. O. R. C.; Capt., Engrs., U. S. A., Feb. 15, 1919. Overseas service Aug. 9, 1917–Apr. 16, 1919. Discharged May 26, 1919. Engr. in chg. field surveying on Bassens and Talmont dock projects and in chg. constr. Bayonne project, Base Sec. No. 2, A. E. F.

CHRISTENSEN, GEORGE ANDREW
Entered service May 19, 1917; Capt., Q. M. R. C., Mar. 21, 1917; Maj., Q. M. C., Constr. Div., U. S. A., Sept. 26, 1918. Constr. work, Camp Kearny; Utilities Officer, Camp Johnston; utilities, Asheville, N. C.; Q. M. duties, Boston, Mass.; utilities and constr., Camp Holabird and Washington, D. C.

CHRISTIE, CHESTER de BAUN
Entered service Sept. 16, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 19, 1918. Camp Humphreys.

CHRISTOPHERS, REGINALD GILLON
2d Lieut., 34th Reinforcements, New Zealand Expeditionary Forces. Died Oct. 13, 1918, from wounds received in action in France.

CHURCH, ELIHU CUNYNGHAM
Maj., Gen. Staff, U. S. A., A. E. F.*

CHURCHILL, PERCIVAL MITCHELL
Entered service May 8, 1917; Capt., E. O. R. C., Mar 1, 1917; Maj., E. O. R. C., Aug. 15, 1917. Overseas service Aug. 26, 1918–June 16, 1919. Discharged July 9, 1919. Bn. Comdr., 304th Engrs.; C. O., 538th Engr. Service Bn.; Adj., 6th Grand Div., T. C., A. E. F.

CHURCHILL, ROBERT CARR
Entered service June. 1917, as 2d Lt. E. O. R. C.; 1st Lt. Engrs., N. A., July, 1918. Overseas service Feb., 1918–Apr., 1919. Discharged Apr. 30, 1919. Constr. work at St. Nazaire, France; light ry. maintenance work in Baccarat Sector; maintenance of light rys. and telephones in Toul Sector. Citation from Gen. Pershing for meritorious service.

CLAFLIN, WILLIAM BEMENT
Entered service May 13, 1917; Capt., E. O. R. C., Sept. 13, 1917. Overseas service Aug. 18, 1918–Jan. 3, 1919. Discharged Jan. 14, 1919. Co. Comdr., 114th Engrs.; Acting Asst. Chf. of Staff, 39th Div., as Intelligence Officer.

CLAIBORNE, HERBERT AUGUSTINE, JR.
Entered service Nov. 8, 1917, as 2d Lt., Aviation Sec., Sig. R. C.; 1st Lt., A. S. A., U. S. A., Oct. 3, 1918. Overseas service, Mar. 4, 1918–May 3, 1919. Discharged May 8, 1919. With 485th Aero Squadron; special duty with Engr. Corps.

CLARK, ERNEST ALDEN
Entered service June 1. 1918; Capt., Engr. R. C., July 1, 1918. Discharged Apr. 3, 1919. With 139th and 209th Engrs.; Engr. Supply Officer, Camp Shelby.

CLARK, HOWARD FOSTER
Entered service Sept. 2, 1917; Capt., E. O. R. C. June 23, 1917; Maj., Engrs. U. S. A., Mar. 28, 1919. Overseas service Jan. 31, 1918–May 6, 1919. With 2d Engrs. at Chateau Thierry; with 7th Engrs. in Vosges; with 314th Engrs. at St. Mihiel; Instr., 3d Corps School, A. E. F. Two stars.

CLARKE, HARRY LEE
Entered service Dec. 28, 1917; 1st Lt., Engrs., N. A., Sept. 26, 1917; Capt., Engrs., U. S. A., May 27, 1919. Overseas service Aug. 29, 1918–July 5, 1919. Discharged July 29, 1919. Co. Comdr. 28th Engrs.; with 1st Army in Meuse-Argonne offensive; rebuilding roads in Meuse Dept. One star.

CLARKE, THOMAS CURTIS†
Entered service May 8, 1917; Capt., E. O. R. C., Jan. 7, 1917; Lt. Col., E. O. R. C., Aug. 5, 1917; Col., Engrs., U. S. A., Aug. 7, 1918. Overseas service Mar. 23, 1918–Mar. 31, 1919. Discharged Apr., 1919. With 110th Engrs.; Acting Deputy Director, A. T. S. Croix de Guerre; Division Citation (French). One star.

† Died May 25, 1921.

CLARKE, WILLIAM DEXTER
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service Mar. 30, 1918–June 9, 1919. Discharged July 3, 1919. Co. Comdr. and Bn. Comdr., 23d Engrs., Meuse-Argonne offensive. One star.

CLARKSON, CHARLES DANA SAYRES
Capt., Engrs., U. S. A.*

CLASS, CHARLES FRANK
Entered service Aug. 9, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 12, 1918. Camp Humphreys.

CLAYTON, THOMAS WILEY
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 23, 1917. Overseas service Jan. 24, 1918–Jan. 25, 1919. Discharged Jan. 29, 1919. With 23d Engrs. at Camp Meade; with 161st Engrs. at Angers, France; ry. constr., Advance Sec., S. O. S., A. E. F.

CLECKNER, FREDERICK MARTIN
Entered service Apr. 17, 1918, as Pvt., Sig. C., N. A. Overseas service Aug. 31, 1918–May 1, 1919. Discharged May 14, 1919. Observer and Recorder of meteorological data at Colombey-les-Belles, Vitrey, and Tours, France.

CLEVELAND, LOU BAKER
1st Lt., Engrs., U. S. A.*

CLINTON, DELMAR SMITH
Entered service Oct. 31, 1917; Capt., E. O. R. C., Apr. 16, 1917; Maj., Engr. R. C., Jan. 14, 1918. Discharged Mar. 11, 1919. With 321st Sappers.

COBB, WILLIAM RICHARD
Lt., U. S. N. R. F.*

COCHRANE, VICTOR HUGO
Entered service Apr. 6, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged Sept. 22, 1919. Asst. to Chf. of Constr. Div., Washington, D. C.; Executive Officer, Ord. Claims Bd., Perth Amboy, N. J.; member, Bd. of Contract Review, and Claims Bd., Constr. Div.

COE, CLARENCE STANLEY
Entered service June 13, 1917, as Capt., E. O. R. C.; Maj., Engr. R. C., Feb. 26, 1918; Lt. Col., Engrs., U. S. A., Feb. 13, 1919; Col., Engrs., U. S. A., Mar. 3, 1919. Overseas service July 28, 1917–Mar. 25, 1919. Discharged Apr. 30, 1919. With 17th Engrs.; Asst. to Sec. Engr., Base Sec. No. 1, A. E. F., in gen. chg. Montoir Project. Legion d'Honneur; Citation from Gen. Pershing for general merit.

COE, DAVID
Entered service Mar. 1, 1917, as Lt., Lines of Communication, Royal Engrs., British Army; Temporary Capt., Apr. 10, 1917. Overseas service Mar. 13, 1917–Apr. 2, 1919. Discharged Apr. 3, 1919. Ry. constr. and maintenance. Mentioned in Sir Douglas Haig's despatches.

COE, EDWARD KIRK
Entered service June 7, 1917; Maj., E. O. R. C., Apr. 18, 1917. Overseas service Nov. 26, 1917–Jan. 1, 1919. Discharged Jan. 4, 1919. Const. Q. M., Camp Lee; Officer in Chg. constr. hosp. center, Nantes, France; with 1st Army, A. E. F. One star.

COLE, HOWARD JUDSON
Entered service July 8, 1918; Capt., Engrs., N. A., June 28, 1918. Discharged Aug. 21, 1919. Historical Data Section, Office of Chf. of Engrs., Washington, D. C.

COLEMAN, EUGENE HUNTER
Entered service May 8, 1917, as 1st Lt., E. O. R. C.; Capt., E. O. R. C., Aug. 15, 1917. Overseas service Aug. 21, 1918–June 25, 1919. Discharged July 16, 1919. Road repair work with 312th Engrs.

COLEMAN, HENRY FITCH
Entered service June, 1917, as Maj., E. O. R. C. Overseas service Nov. 23, 1917–Nov., 1918. Discharged Feb. 3, 1919. With French Artillery at Camp du Meucon; convoyed munitions to British ports; convoyed replacements Angers to Ardennes, France.

COLGAN, ROBERT JOSEPH
Capt., Engrs., U. S. A., A. E. F.*

COLHOUN, DANIEL WARWICK
Entered service May 26, 1917; Capt., Engr. R. C., Jan. 3, 1918; Maj., Tank C., U. S. A., Nov. 8, 1918. Overseas service July, 1917–Mar. 1919. Co. Comdr., 1st Engrs.; with 305th Brig.; Eng. Intelligence Office, G. H. Q. Divisional citation. Three stars.

COLLIER, WILLIAM NEVILLE
Entered service Sept. 11, 1918, as Capt., Engrs, U. S. A. Discharged Nov. 30, 1918. Camp Humphreys.

COLLINS, EARL HARRY
Entered service Aug. 15, 1917, as Capt., C. A. R. C.; Capt., Engrs., U. S. A., Sept. 26, 1918. Discharged Mar. 20, 1919. In chg. constr. work at Ft. Monroe, Va.

COLLINS, MERTON CLYDE
Entered service June 28, 1917, as 1st Lt., E. O. R. C. Discharged Dec. 18, 1918. Unattached, Philippine Islands; with 565th Engr. Service Bn., Camp Shelby.

COLUMBIA, CURTIS FIELDS
Entered service May 1, 1917; 1st Lt., C. A. C., N. A., Aug. 14, 1917; Capt., C. A. C., U. S. A., Sept. 22, 1918. Overseas service Sept. 12, 1917–Mar. 20, 1919. Discharged Mar. 21. 1919. With 52d Artillery; Asst. Supervisor of Instruction, Saumur Artillery School, France. One star.

COMBER, STAFFORD XAVIER
Entered service May 9, 1918. 2d Lt., British Guiana Inf. Resigned Nov., 1918. Served in British Guiana supplying contingents to British West Indies Regt. for France and Egypt.

COMLY, HARRY SEYKORA
Entered service Mar. 13, 1918, as Pvt., Sig. C., N. A.; Sgt., Sig. C., U. S. A., Nov., 1918. Discharged Jan. 29, 1919. In Science and Research Div., calculation of amended ballistic tables for atmospheric corrections.

COMLY, JAMES RETZER
Entered service Dec. 28, 1917; 1st Lt., Engrs., N. A., Aug. 1, 1917. Discharged Nov. 29, 1918. Organization and Co. Comdr., engr. units at Camp Humphreys and Ft. Myer; with 99th Engrs.

COMPTON, ARTHUR MANDEVILLE
Entered service June 21, 1917, as Maj., F. A. R. C.; Lt. Col., F. A. R. C., Aug. 1, 1917; Col., F. A., N. A., July 30, 1918. Discharged Dec. 15, 1918. With 126th F. A.; Ft. Sill, student and Instr., School of Fire; 31st F. A.; Acting Brig. Comdr., 11th F. A.; Comdr., 32d F. A.

COMPTON, RUEL KEITH
Entered service July 21, 1917, as Maj., Engrs., N. A. Discharged June 6, 1919. In chg. bldgs. and roads, Camp Meade; in chg. bldgs., roads, water, sewerage, docks and warehouses. Curtis Bay Ord. Depot, Md.; Executive Officer, Little Rock Picric Acid Plant; Acting Const. Q. M., Little Rock Picric Acid Plant.

CONARD, CLARENCE KNIGHT
Entered service Dec. 8, 1917, as Maj., Ord. R. C., N. A. Discharged Oct. 30, 1919. Detailed to Constr. Div., U. S. A., as Const. Q. M., Raritan Arsenal.

CONARD, ROBERT ALLEN
Entered service Sept. 8, 1917, as Lt., C. E. C., U. S. N. R. F. Discharged June 26, 1920. Served in Haiti on public works, chiefly road building.

CONNELLY, WALTER LOUIS
M. E., Sr. Grade, Engrs., U. S. A., A. E. F.*

CONNOLLY, ALLEN HOWARD
Entered service Feb. 28, 1918, as Pvt., 1st Class, Aviation Sec., Sig. R. C.; 2d Lt., A. S. A., U. S. A., May 2, 1918. Discharged Jan. 2, 1919. School of Military Aeronautics, Columbus, Ohio; 34th Constr. Squadron; with Govt. Field Office, constr. work, Langley Field, Hampton, Va.

CONNOLLY, DONALD HILARY
Entered service June 15, 1906; through all grades in C. of E., U. S. A., to Maj., Aug. 5, 1917; Lt. Col., May 23, 1918; Col., Engrs., U. S. A., Aug. 1, 1919; Maj., C. of E., U. S. A. (permanent rank), June 1, 1920. Overseas service June 4, 1918–Sept. 19, 1918. Co. Comdr., El Paso, Tex.; in chg. div. schools, 30th Div.; Asst. Chf. of Staff (G–3), 8th Div.; G. S. C., U. S. A.; Office, Chf. of Staff, Washington, D. C. One star.

CONNOR, WILLIAM DURWARD
Entered service June 21, 1893; through all grades in C. of E., U. S. A., to Col., Aug. 5, 1917; Brig. Gen., U. S. A., June 26, 1918. Overseas service July 14, 1917–Jan. 24, 1920. Organized G-4, G. H. Q., A. E. F.; Chf. of Staff, 32d Div.; in command 63d Inf. Brig. and S. O. S., Base Section No. 2, A. E. F.; Chf. of Staff, S. O. S., A. E. F.; in command American Forces in France from Sept. 1, 1919. Distinguished Service Medal; Commandeur, Legion d'Honneur; Companion of the Order of the Bath, Great Britain.; Croix de Guerre, with Palm; Order of the Solidaridad, Panama. Two stars.

CONVERSE, JOSEPH BRANDLY
Entered service Aug. 18, 1918, as 2d Lt., Q. M. C., Constr. Div., U. S. A. Discharged Feb. 6, 1919. Attached to camp utilities as officer in chg. of operations.

CONWAY, NORMAN BUTLER
Entered service July 5, 1917, as Capt., E. O. R. C. Overseas service June 10, 1918–June 28, 1919. Discharged July 25, 1919.

COOK, PAUL DARWIN
Entered service May 5, 1917, as Capt., E. O. R. C.; Maj., Engrs., U. S. A., Oct. 5, 1918. Discharged Mar. 22, 1919. Co. Comdr., 109th Engrs., Camp Cody; Adj., 8th Engr. Training Regt., Camp Humphreys; Bn. Comdr., 75th and 138th Engrs.

COOKE, FREDERICK HOSMER
Entered service Jan. 1, 1904; through all grades in C. E. C., U. S. N., to Comdr., July 1, 1918. Overseas service Apr. 16, 1918–Jan. 19, 1919. In chg. constr. Lafayette Radio Station near Bordeaux, France.

COOKSEY, ROBERT MAVIN
Entered service July 22, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged Mar. 24, 1919.

COOMBS, ROBERT DUNCAN
Entered service Nov., 1917; Maj., Ord. R. C., N. A., Dec. 8, 1917. Discharged May 17, 1919. Staff duty, Nitrate Div.; Special Aide on Nitrate Plant No. 2; Chf. of Sec., Plant No. 2; Officer in chg. and agent of Cont. Officer, Alabama Power Co. Contract.

COOMER, ROSS MILLER
Entered service May 7, 1917, as Capt., E. O. R. C.; Maj., Engr. R. C., May 13, 1918; Lt. Col., Engrs., U. S. A., Apr. 9, 1919. Overseas service Aug. 14, 1917–May 17, 1919. Discharged May 20, 1919. Engr. Officer in chg. of Hospitalization, Chf. of Bldg. Sec., Div. of Constr. and Forestry, S. O. S., A. E. F.

COOPER, DAVID REGINALD
Entered service Sept. 26, 1917; Capt., Engrs., N. A., Aug. 16, 1917. Overseas service Oct. 3, 1917–Jan. 3, 1919. Discharged Feb. 6, 1919. Designed, built, and operated water supply and purification plants, A. E. F.; designed and built concrete arched dam at Savenay, France.

COOPER, HUGH LINCOLN
Entered service June 28, 1917, as Maj., E. O. R. C.; Lt. Col., E. O. R. C., Oct. 26, 1917; Col., Engr. R. C., Mar. 20, 1918. Overseas service July 14, 1917–Mar. 3, 1918, and May 14, 1918–Oct. 17, 1918. Discharged Nov. 7, 1918. Cantonment work in U. S.; Sec. Engr., Base Sec. No. 2, Bordeaux, France.

COPELAND, ROBERT MORRIS
Entered service May 8, 1917; 2d Lt., E. O. R. C., Feb. 17, 1917; 1st Lt., Engrs., N. A., June 11, 1918; Capt., Engrs., U. S. A., Oct. 9, 1918. Overseas service Dec. 11, 1917–Apr. 13, 1918. Organization work with 314th Engrs., Camp Funston; with 1st Engrs. in Toul Sector; with 315th and 111th Engrs., Special Instr.; Office, Chf. of Engrs., Personnel Sec.

CORBIN, HORACE KELLOGG
1st Lt., Q. M. C., U. S. A.*

CORKRAN, WILBUR SHERMAN
Entered service May, 1917, as 1st Lt., E. O. R. C.; Capt., Engrs., U. S. A., Jan., 1918; Maj., Engrs., U. S. A., Aug. 1918. Overseas service July, 1917–Aug., 1918. Discharged May 22, 1919. Co. Comdr., 1st Engrs.; Staff and Engr. Officer, T. S., Camp Humphreys.

CORNELL, JOHN WESLEY
Entered service Feb., 1918, as Capt., Q. M. C., N. A. Discharged July 17, 1919. On constr., Brooklyn Army Supply Base.

CORNISH, LORENZO DANA
Entered service June 5, 1917; Capt., E. O. R. C., June 20, 1917; Maj., Engrs., U. S. A., Feb. 13, 1919. Overseas service, July 9, 1917–June 18, 1919. Discharged Oct. 30, 1919. With 15th Engrs. in constr. work, Bassens Dock, Ord. Depot No. 4, Advance Ord. Depot No. 4, A. E. F.; in light ry. work in shops and along front; salvaging light ry. in Argonne; road repair work. Two stars.

CORP, HENRY WILLIAM
Entered service June 20, 1918, as 1st Lt., Engrs., N. A. Discharged Jan. 14, 1919. Co. Comdr., A. R. D.; with 2d Engr. Training Regt.; Asst. Judge Advocate, Gen. Court No. 2, Camp Humphreys.

CORY, HARRY THOMAS
Maj., Engrs., U. S. A.*

COSBY, SPENCER
Entered service 1887; through all grades in C. of E., U. S. A., to Col., Aug. 5, 1917. Overseas service Aug. 1, 1914–Feb. 1, 1917. Military Attaché, American Embassy, Paris. Aide to Marshal Joffre; C. O., 5th, 605th and 209th Engrs. Officier, Legion d'Honneur.

COTTER, CARL HENRY
Entered service Dec. 27, 1917, as Lt., Jr. Grade, C. E. C., U. S. N.; Lt., C. E. C., U. S. N., July 1, 1918. Duty at U. S. Naval Academy; Navy Yard, Washington, D. C.; special duty under Engr. in Chf., Haiti.

COTTMAN, LEWIS WARRINGTON
Entered service Aug. 18, 1918, as Maj., C. W. S., U. S. A. Discharged Feb. 20, 1919.

COURTNEY, MAURICE ALDEN
Entered service Mar. 4, 1918, as 2d Lt., T. C., N. A.; 1st Lt., T. C., N. A., Feb. 12, 1918. Overseas service, Apr. 25, 1918–July 7, 1920. Discharged Aug. 22, 1920. In chg. troop movements, Advance Sec., A. E. F.; with Inter-Allied War Reparations Bd., Germany; G-4, Transportation H. Q., American forces in Germany.

COVERT, JOHN FRANCIS
Entered service July 24, 1917; 1st Lt., E. O. R. C., June 13, 1917. Discharged Aug. 29, 1918. Asst. Dist. Engr., Big Bend Dist., Texas; 111th Engrs., Camp Bowie; Engr. Training Camp, Camp Lee; with 55th Engrs., Camp Custer.

COWLES, WILLIAM PIERCE
Entered service Dec. 28, 1917; Maj., E. O. R. C., Sept. 19, 1917. Overseas service July 8, 1918–Jan. 6, 1919. Discharged Jan. 6, 1919. Organized 3d Bn., 34th Engrs.; C. O., 3d Bn., 34th Engrs., Intermediate Engrs. Dept., Gierves, France.

COX, LEONARD MARTIN
Entered service Feb., 1899; through all grades in C. E. C., U. S. N., to Comdr. Public Works Officer, 12th Naval Dist.; Asst. Mgr., Div. Shipyard Plants, Emergency Fleet Corp.; in chg. constr., Air Station Training Camp, San Diego, Cal.; dock operations, Emergency Fleet Corp.

COY, BURGESS GREENACRE
Entered service May 14, 1917; Capt., Engrs., N. A., July 12, 1917; Maj., Engrs., U. S. A., Mar. 26, 1919. Overseas service June 12, 1918–May 26, 1919. Discharged June 20, 1919. With 314th Engrs. in U. S. and France.

CRAGIN, CHARLES CALHOUN
Entered service June 13, 1917, as Capt., E. O. R. C.; Maj., Engrs., U. S. A., Aug. 8, 1918; Lt. Col., Engrs., U. S. A., May 26, 1919. Overseas service Aug. 9, 1917–Aug. 2, 1919. Discharged Aug., 1919. Co. Comdr., 18th Engrs.; C. O., 130th Engrs.; in chg. constr., hosp., railroad and engine terminal, Base Sec. No. 2, A. E. F.; Chf. Engr., Base Sec. No. 2 and No. 7.

CRANDALL, CARL
Entered service Dec. 14, 1917; Flying Cadet, Cornell Univ., Feb. 16, 1918; 2d Lt., A. S. A., U. S. A., Oct. 22, 1918. Discharged Dec. 31, 1918. In training Dorr Field, Arcadia, Fla., Barron Field, Fort Worth, Tex., and Brooks Field, San Antonio, Tex.

CRANE, ALBERT ELI
Entered service May 1, 1917; 1st Lt., E. O. R. C., Feb. 19, 1917. Capt., Engrs., U. S. A., July 20, 1918. Overseas service Dec. 4, 1917–Apr. 29, 1919. Discharged May 19, 1919. Regtl. Adj., 6th Engrs. Four stars. One wound.

CRANE, JOSEPH SPENCER
Entered service Apr. 24, 1918, as Maj., Q. M. C., Constr. Div., N. A. Officer in chg. maintenance and repair of all bldgs. at camps and Regular Army Posts; Administrative Div., military intelligence; with Director of Storage and Purchase; Const. Q. M., temporary Motor Storage Depot, South Amboy, N. J.

CRAWFORD, HUGH WILLIAM
Entered service Apr. 5, 1917, as Pvt., Kansas N. G.; 1st Lt, Engr. N. G., Apr. 14, 1917; Capt., Engr. N. G. Aug. 4, 1917. Resigned Mar. 4, 1918, and enlisted in Naval Reserve, Mar. 20, 1918; Ensign, U. S. N. R. F., Oct. 5, 1918. Released from active duty Mar. 9, 1919. With 1st Kansas Inf., Kansas Engrs., 110th Engrs.; Jr. Officer, U. S. S. *Absaroka*.

CRAWFORD, IVAN CHARLES
Entered service Aug. 5, 1917, as Capt., Engrs., N. A.; Maj., Engrs., N. A., Oct. 21, 1917. Overseas service Aug. 8, 1918–June 16, 1919. Discharged June 19, 1919. Bn. Comdr., 115th Engrs.; Chf., Gen. Bldg. Sec., Belgian Mission, War Damages Sec., American Comm. to Negotiate Peace.

CRECELIUS, SAMUEL FORDER
Entered service Sept. 2, 1917; Maj., E. O. R. C., June 28, 1917; Lt. Col., Engr. R. C., Feb. 15, 1918; Col., Engrs., N. A., July 30, 1918. Overseas service Sept. 26, 1917–Aug. 23, 1918. Discharged Jan. 23, 1919. In chg. Personnel Sec., Office Chf. Engr., A. E. F.; with 101st Engrs., officer in chg. field fortifications and road repair, Chateau Thierry; in chg. Pioneer School, Camp Humphreys.

CRENSHAW, BERNARD LEE
Entered service July 8, 1918, as Pvt., Engrs., N. A.; Sgt., Engrs., N. A. Overseas service Aug. 22, 1918–Apr. 29, 1919. Discharged May 16, 1919. With 22d Engrs., in light ry. work at front in Toul Sector and Meuse-Argonne operations. One star.

CROCKER, HERBERT SAMUEL
Entered service Oct. 20, 1917; Maj., E. O. R. C., June 28, 1917; Lt. Col., Q. M. C., Constr. Div., U. S. A., Aug. 24, 1918. Discharged Oct. 23, 1919. One of two Superv. Const. Q. M., Port Newark Terminal, Raritan River Ord. Depot, and Camps Stuart and Hill; Superv. Const. Q. M., Middletown, Pa., Ord. Depot and Raritan River Ord. Depot; Const. Q. M., Army Supply Base, Brooklyn, N. Y.

CROCKER, JAMES ROGER
Entered service Feb. 23, 1917; Capt., E. O. R. C., Sept., 1917; Maj., Engrs., U. S. A., Sept., 1918. Overseas service Sept. 16, 1918–Aug. 31, 1919. Discharged Sept. 19, 1919. Co. Comdr., Engr. Train, 306th Engrs.; Co. Comdr. and Bn. Comdr., 106th Engrs.; in chg. constr., Kerhuon project, France; Executive Officer, Chf. Engr., Base Sec. No. 5. One star.

CROMWELL, GEORGE
Entered service Sept. 2, 1918, as Capt., Engrs., N. A. Discharged Dec. 11, 1919.

CRONEMEYER, JOHN FREDERICK WILLIAM
Entered service Nov. 1, 1918; Student, E. O. T. S. Discharged Nov. 27, 1918. Camp Humphreys.

CROOKS, CLINTON HERVEY
Entered service July 13, 1918, as Capt., Engrs., N. A. Discharged Dec. 4, 1918. Gen. constr.; with 551st Labor Bn., Camp Humphreys.

CROSBY, LOTHROP
Entered service Sept. 23, 1918; 1st Lt., E. O. T. S. Discharged Jan. 11, 1919. Camp Humphreys.

CROSBY, WALTER WILSON
Entered service Aug. 24, 1917, as Maj., Engrs., N. A.; Lt. Col., Engrs., N. A., Sept. 7, 1917. Overseas service June 17, 1917–Oct. 12, 1919. Discharged June 30, 1919. With 104th Engrs. as 2d in command; at H. Q., 7th Army Corps; Office, Chf. Engr., A. E. F. Citation from Gen. Pershing for meritorious service. Letter of commendation from Maj. Gen. Langfitt. One star.

CROSSON, WILLIAM HENRY
Entered service May 8, 1917; 1st Lt., E. O. R. C., June 13, 1917; Capt., Engrs., U. S. A., Apr. 8, 1919. Overseas service Nov. 11, 1917–Aug. 4, 1919. With 20th, 128th, and 2d Engrs.

CROWELL, FRANCIS STIRLING
Entered service June 25, 1918, as Capt., Q. M. C., Constr. Div., N. A. Discharged Aug. 31, 1919. Asst. to Const. Q. M., Army Supply Base, Brooklyn, N. Y., in chg. constr. piers and bulkhead.

CUDEBEC, ALBERT BENNETT
Entered service July 28, 1917, as Capt., Engrs., N. A.; Maj., Engrs., U. S. A., Feb. 13, 1919. Overseas service Aug. 17, 1917–May 30, 1919. Discharged June 17, 1919. On Technical Staff, Director Gen. Transportation, A. E. F.; Chf. of Technical Bd., G. P. B.; Liaison officer between U. S., French, and British Engrs.; U. S. Liquidation Comm. Chevalier, Legion d'Honneur; Officier d'Academie, with Silver Palms. One star.

CULVER, ARTHUR
Lt., Royal Engrs., B. E. F.*

CUNLIFF, CHARLES, JR.
Entered service Mar. 18, 1918; 2d Lt., Engrs., N. A., June 3, 1918. Discharged Dec. 13, 1918. With 32d Engrs.; with 54th Engrs., Camp Dix; 403d Engrs., Camp Douglas; in command E. O. T. S., Ft. Douglas, Utah.

CUNNINGHAM, JOHN GEORGE LAWRENCE
Capt., Ord. Dept., U. S. A.*

CURFMAN, LAWRENCE EVERETT
Entered service May 8, 1917; Capt., E. O. R. C., Jan. 23, 1917; Maj., E. O. R. C., Aug. 15, 1917; Lt. Col., Engrs., N. A., June 12, 1918. Overseas service June 12, 1918–Jan. 13, 1919. Discharged Jan. 17, 1919. With 314th Engrs., as Bn. Comdr., and 2d in command.

CURLEY, JAMES FRANCIS
Entered service Sept. 4, 1918, as Maj., Ord. Dept., U. S. A. Discharged Dec. 6, 1918. Office, Chf. of Ord., U. S. A., Chf. of Progress Sec.; Production Div.; Purchase, Storage and Traffic Div., General Staff.

CURREY, JOHN WAGGONER
Entered service May 8, 1917; 1st Lt., E. O. R. C., May 16, 1917. Overseas service Aug. 8, 1918–Sept. 26, 1919. Discharged Oct. 2, 1919. Dist. Engr. Officer, Laredo, Tex.; with 115th Engrs. at Camp Kearny; with Chf. Engr., 3d Army, A. E. F. One star.

CURRIE, THOMAS AUSTIN, JR.
Sgt., Engrs., U. S. A., A. E. F.*

CURTIS, VARNUM PIERCE
Capt., Engrs., U. S. A.*

CUSHING, EDWARD BENJAMIN
Entered service June 14, 1917, as Maj., E. O. R. C.; Lt. Col., T. C., N. A., Feb. 28, 1918; Col., T. C., U. S. A., Apr. 18, 1919. Overseas service July 28, 1917–Aug. 8, 1919. Discharged Sept. 28, 1919. Supt., A. T. S., La Rochelle Dist. and Marseilles, France; Gen. Supt. Transportation, Mediterranean ports; Dept. Dir., A. T. S., Base Sec. No. 9; Member various boards investigating base ports for American forces, Inter-Allied Comm. for the transportation of supplies to the allied armies in Germany. Two citations for meritorious service; Legion d'Honneur. Two stars.

CUTLER, LEON GEORGE
Entered service Aug. 25, 1917; Capt., C. A. R. C., Nov. 27, 1917. Overseas service May 10, 1918–Mar. 25, 1919. Discharged Mar. 30, 1919. With 57th Regt., C. A. C., in St. Mihiel and Meuse-Argonne offensives. Three stars.

DAAE, HANS ANDREAS
Entered service Nov. 4, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 10, 1918.

DALLIS, PARK ANDREW
Capt., Engrs., U. S. A.*

DALY, ALBERT PETER VINCENT
2d Lt., Connaught Rangers, B. E. F.*

DALY, DAVID AUGUSTUS
Capt., Engrs., U. S. A., A. E. F.*

DALY, JOHN WILLIAM
Entered service Aug. 23, 1917; 2d Lt., Inf., N. A., Nov. 23, 1917; 1st Lt., Inf., U. S. A., Sept., 1918. Overseas service Oct. 1918–Mar., 1919. Discharged Mar., 1919.

DANFORTH, GEORGE CLAPP
Entered service Dec. 1917; Capt., Engrs., N. A., Sept. 26, 1917. Overseas service July 9, 1918–June 27, 1919. Discharged July 30, 1919. 3d E. O. T. C., Camp Lee; with 602d Engrs., Camp Humphreys. Camp Devens, and overseas; with 1st Army, A. E. F., St. Mihiel and Meuse-Argonne offensives. Three stars.

DARLING, CLINTON STOWELL
Entered service July 27, 1917, as Pvt., Engrs., N. A.; Cpl., Engrs., N. A., Aug. 9, 1917; Sgt., Engrs., N. A., Sept. 26, 1917; 2d Lt., Engrs., N. A., Dec. 25, 1917; 1st Lt., Engrs., N. A., Apr. 19, 1918; Capt., Engrs., U. S. A., Sept. 14, 1918. Overseas service May 8, 1918–Sept. 29, 1919. Discharged Oct. 17, 1919. Co. duty with 108th Engrs., Topographic Officer. Three stars.

DARVILLE, MERTON ARTHUR
2d Lt., Engrs., U. S. A., A. E. F.*

DAVIDSON, GEORGE BURRETT
Entered service Sept. 2, 1917; 2d Lt., Engrs., N. A., July 24, 1917. Overseas service July 10, 1918–Aug. 4, 1919. Discharged Aug. 29, 1919. With 29th Engrs.; H. Q., 26th Div., A. E. F.; Topographic Officer.

DAVIES, JOHN PERCIVAL
Entered service Jan. 17, 1918, as Capt., Ord., N. A. Discharged Jan. 7, 1919. Acting Adj., Drop-Bomb Unit, Aircraft Armament Sec., Ord. Corps.

DAVIS, CHANDLER
Entered service May 8, 1917; Capt., E. O. R. C., Jan. 23, 1917; Maj., Engrs., U. S. A., Apr. 9, 1919. Overseas service Dec. 4, 1917–Sept. 17, 1919. Discharged Oct. 5, 1919. With 6th Engrs. as Topographic and Co. Officer; C. O., 1st Engrs. Casual Co.; with Sec. Engr., Base Sec. No. 1, on English front and in St. Mihiel and Meuse-Argonne offensives. Military Cross, Great Britain. Two stars. One wound.

DAVIS, DANIEL ELIAS
Entered service May 13, 1917; 1st Lt., E. O. R. C., June 13, 1917; Capt., Q. M. C., Constr. Div., Mar. 28, 1918. Discharged Oct. 25, 1919. Asst. Const. Q. M. at Camp Hancock, Baltimore Depot Warehouses; Const. Q. M. and Utilities Officer.

DAVIS, FREDERICK AUGUSTUS WILLIAM
Entered service June 22, 1917; 1st Lt., Engrs., U. S. A., July 21, 1917; Capt., Engrs., U. S. A., July 30, 1918. Overseas service May 17, 1918–Sept. 7, 1918. Discharged Jan. 14, 1919. With 102d Engrs. and 3d British Corps, Ypres-Kemmel, Belgium, in chg. operations M. T. Service, Camp Humphreys; Asst. Div. Engr., Camp Lewis. Two stars.

DAVIS, JOHN CHARLES
Entered service May 8, 1917; Capt., C. A. C., N. A., Aug. 15, 1917. Discharged Dec. 24, 1918. With C. A. C. at Ft. Monroe; with 40th Artillery.

DAVIS, ROBERT MENEES
Entered service Dec. 28, 1917; 1st Lt., E. O. R. C., July 20, 1917; Capt., Engrs., N. A., July 13, 1918. Overseas service Aug. 1, 1918–May 29, 1919. Discharged June 3, 1919. Co. Comdr., 57th Engrs.; Supt. of Ports at Villeveuve le Roi and Grigny, France, Inland Waterway Transport.

DAVY, JESSE JOHN
Entered service May 8, 1917; 1st Lt., E. O. R. C., July 11, 1917; Capt., Engrs., U. S. A., Sept. 27, 1918. Overseas service Jan. 23, 1918–June 9, 1919. Discharged July 8, 1919. Co. Comdr., 23d Engrs.; on staff of Chf. Engr., 1st Army, A. E. F.; Toul Sector, Chateau-Thierry, Vesle, St. Mihiel and Meuse-Argonne operations. Three stars. Wounded at Romagne-sous-Montfaucon Oct. 27, 1918.

DAY, WARREN ELLIS
Entered service Dec. 28, 1917; Capt., E. O. R. C., Sept. 26, 1917. Overseas service July 10, 1918–July 17, 1919. Discharged Aug. 13, 1919. Co. Comdr., 45th Engrs.; C. O., T. C. troops, Camp Montoir, France.

DEAN, BERTRAM DODD
Entered service May 10, 1917; Capt., E. O. R. C., June 13, 1917; Acting Maj., Engrs., U. S. A., Nov. 12, 1918. Overseas service Feb. 28, 1918–May 22, 1919. Discharged July 8, 1919. Co. Comdr., Supply Officer and Regtl. Adj., 25th Engrs.; special duty with Hospitalization Section, Office, Chf. Engr., A. E. F.; in chg. hosp. constr. at La Suze, France; constr. and maintenance of roads; Meuse-Argonne offensive. One star.

DEAN, STANLEY
Entered service Aug. 24, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Superv. Engr., with Const. Q. M., Camp Humphreys. Died Oct. 2, 1918, in active service at Camp Humphreys, Va.

DEAN, WILLIS JOHNSON
Entered service May 11, 1918, as 1st Lt., Q. M. C., Constr. Div., N. A.; Capt., Q. M. C., Constr. Div., U. S. A., Oct. 22, 1918. Discharged May 15, 1919. On duty at Washington, D. C.; Asst. to Const. Q. M., Army Supply Base, Brooklyn, N. Y.; in chg. design and constr. additions to Benicia, Cal., Arsenal; Asst. to Chf. of Production Sec., Eng. and Standardization Branch, Gen. Staff, Washington, D. C.

de CHARMS, RICHARD, JR.
Entered service May 8, 1917; 1st Lt., Engrs., N. A., Sept. 8, 1917. Overseas service Nov. 26, 1917–June 9, 1919. Discharged July 8, 1919. With 503d and 21st Engrs.; special duty as Chf. Draughtsman, Dept. Railroads and Docks, Constr. and Forestry Div., Office, Chf. Engr., A. E. F.; Toul Sector, St. Mihiel and Meuse-Argonne offensives. Two stars.

DECKER, ARTHUR JAMES
Entered service Oct. 22, 1918, as Capt., San. C., U. S. A. Discharged Jan. 4, 1919. Camp San. Engr., Camp Meade.

DEDICKE, ERNEST CHARLES
Entered service 1917 as 2d Lt., Engrs., U. S. A.; 1st Lt., Engrs., U. S. A., 1917. Overseas service Nov., 1917–July 15, 1918. E. O. T. C., Madison Barracks; with 6th Engrs.; Engr. Officer with 4th Inf. Died July 15, 1918, from wounds received in action at Chateau Thierry, France.

DEETS, EDWARD HENDERSON
Entered service Aug. 5, 1917; 1st Lt., E. O. R. C., June 23, 1917; Capt., Q. M. C., Constr. Div., N. A., Mar. 18, 1918. Discharged Sept. 2, 1919. Asst. to Const. Q. M., Camp Kearny, Ft. Rosecrans, Ft. Mason, Ft. H. G. Wright; Const. Q. M., Chapman Field, Fla.

DE GARMO, ROBERT MAX
Entered service July 21, 1917, as 1st Lt., Engrs., N. A.; Capt., Engrs., N. A., Feb. 28, 1918; Maj., Engrs., U. S. A., Mar. 4, 1919. Overseas service July 27, 1917–Mar. 25, 1919. Discharged Apr. 12, 1919. With 17th Engrs.; in chg. constr. docks and bridges, St. Nazaire, France. Citation from Gen. Pershing for meritorious service.

DE GRAFF, CARLTON ROBB
1st Lt., Engrs., U. S. A.*

DEISER, NORMAN ARTHUR
2d Lt., F. A., U. S. A.*

DELAMATER, STEPHEN TRUESDELL
Entered service Oct. 27, 1918, as Capt., Q. M. C., Constr. Div., U. S. A.; Maj., Q. M. C., Constr. Div., U. S. A., June 8, 1919. Discharged Oct. 31, 1919. In chg. Bldg. Sec., Constr. Div.; Superv. Const. Q. M. at seven army camps.

DE LEUW, CHARLES EDMUND
Entered service May 10, 1917; 1st Lt., Engrs., N. A., July, 1917; Capt., Engrs., U. S. A., Aug. 10, 1918. Overseas service Apr. 29, 1918–Jan. 19, 1919. Discharged Jan. 22, 1919. With 4th Engrs.; Topographical Officer, Div. H. Q.; Aisne-Marne, St. Mihiel, Meuse-Argonne and Vesle operations. Distinguished Service Cross; Order of the Crown, Belgium. Four stars. One wound.

DE MOSS, SAMUEL
Entered service Apr. 19, 1918; as Pvt., A. S., N. A.; Sgt., Sig. C., U. S. A., Nov. 5, 1918. Discharged Dec. 15, 1918. On duty in Meteorological Sec.

DENT, ELLIOTT JOHNSTONE
Entered service June 19, 1897; through all grades in C. of E., U. S. A., to Col., Aug. 31, 1917. Overseas service June 23, 1918–Mar. 25, 1919. Organized 26th Engrs. and started organization of 24th and 34th Engrs.; C. O., 104th Engrs.; Div. Engr., 29th Div.; C. O., 4th Engrs.; Div. Engr., 4th Div. Citation for meritorious and conspicuous services. Two stars.

DERBY, GEORGE McCLELLAN
Entered service May 25, 1874; through all grades in C. of E., U. S. A., to Col., May 17, 1919. Retired Aug. 12, 1919. In chg. 4th Dist., Mississippi River.

DIESEM, HARRY CUSTER
Entered service May 1, 1918; Capt., Engr. R. C., Jan. 26, 1918. Discharged Oct. 28, 1919. Officer in chg. Dept. Eng. Depot, Ft. Leavenworth; C. O., 404th Engrs., Depot Detachment.

DIFFENDERFER, CLAUDE ORVILLE
Entered service May 14, 1917; Capt., Engrs., N. A., Aug. 14, 1917. Overseas service Aug 29, 1918–July 4, 1919. Discharged July 14, 1919. With 313th and 534th Engrs.

DILLMAN, GEORGE LINCOLN
Entered service Dec., 1917; Maj., Engrs., N. A., Mar. 17, 1918. Overseas service July 10, 1918–Jan. 21, 1919. Discharged Jan. 31, 1919. With 45th Engrs.; Engr., M. of W., standard gauge lines, S. O. S., A. E. F.

DOBSON, GILBERT COLFAX
Entered service May 8, 1917; Capt. E. O. R. C., May 5, 1917; Maj., Engrs., U. S. A., July 30, 1918. Overseas service June 12, 1918–Aug. 26, 1918. With 314th Engrs. One star.

DODD, JOSEPH HOLMES LEE
2d Lt., British West Indies Regt.*

DOEBLER, VALENTINE SHERMAN
Entered service July 6, 1917; Capt., Q. M. R. C., May 22, 1917. Discharged Dec. 10, 1918; Const. Q. M., Ft. Meyer and at Williamsburg, Va.; Asst. Advisory Engr., camp transportation facilities, Constr. Div., U. S. A.

DOELEMAN, HERMAN FRANCOIS
Entered service Apr. 13, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged May 31, 1919. Special assignments in eng.; designed Camps Holabird, Perry, and Atlanta; Chf. of Steel Unit, Procurement Sec.; Sec. Chf., Bldg. Materials Sec., Washington, D. C.

DOHM, EDWARD CLARENCE
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service Sept. 24, 1918–July 5, 1919. Discharged Aug. 1, 1919. C. O., 4th Engrs., Vancouver Barracks; C. O., detachment 2d Engrs.; in chg. Engr. Depot, Camp Humphreys; Engr. Officer, Camp Humphreys; C. O., 471st Engrs.; Co. Comdr., 605th Engrs.; with 29th Engrs. in front area surveys. One star.

DONAHEY, JOSEPH ALEXANDER
Entered service Aug. 20, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 23, 1918. With 147th Engrs.

DONALDSON, CARL S.
Pvt., Officers Training School, Heavy Artillery, U. S. A.*

DONLEY, WILLIAM McCLURG
Entered service May 7, 1918; Capt., E. O. R. C., May 3, 1918. Discharged Feb. 1, 1919. Co. Comdr. with Replacements for 1st Engrs.; in chg. gen. eng. field constr.

DONNELLY, JOSEPH FRANCIS SINNOTT
Entered service Nov. 5, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 18, 1918. Duty with troops at Ft. Benjamin Harrison; with 32d Co. Engrs., unassigned; Asst. Personnel Officer.

DOTEN, LEONARD SMITH
Entered service June 1, 1917, as Capt., Q. M. C., N. A.; Maj., Q. M. C., Constr. Div., N. A., Mar. 18, 1918.

DOUGLASS, LOUIS REA
Entered service May 17, 1917; Capt., Q. M. C., N. A., Mar. 7, 1917. Discharged June 24, 1919. In chg. sewers and sewage disposal, cantonments, Leon Springs, Tex.; Const. Q. M., Leon Springs; in chg. constr. U. S. Gen. Hosp. No. 7, Baltimore; Const. Q. M., Henry Ford Hosp.

DOW, HEZEKIAH SHAILER
Entered service Nov. 14, 1917; 2d Lt., A. S., N. A., Nov. 12, 1917; 1st Lt., Engrs., U. S. A., Nov. 10, 1918. Overseas service Nov. 22, 1917–Jan. 4, 1919. Discharged Jan. 17, 1919. Supply Officer, H. Q., A. S., S. O. S., A. E. F.; Asst. to officer in chg. Hosp. Sec., Dept. of Constr. and Forestry; Office, Chf. Engr., A. E. F.

DOWNING, CARL E.
1st Lt., Engrs., U. S. A.*

DOYLE, JOHN STEPHEN
Entered service June 6, 1918; Capt., E. O. R. C., Jan. 19, 1917; Maj., Engrs. N. A., June 22, 1918. Discharged Jan. 17, 1919. Rivers and Harbors Sec., Office, Chf. of Engrs., Washington, D. C.

DRAYTON, NEWBOLD
Entered service Aug. 14, 1917, as Pvt., F. A., N. A.; Cpl., F. A., N. A., Apr., 1918; 2d Lt., F. A., U. S. A., Apr. 18, 1919. Overseas service May 19, 1918–May 22, 1919. Discharged May 24, 1919. With 108th F. A. Two stars.

DREW, CHARLES DAVIS
Entered service May 13, 1917; Capt., E. O. R. C., June 18, 1917; Maj., Engrs., U. S. A., Apr. 7, 1919. Overseas service July 14, 1917–Apr. 26, 1919. Discharged May 5, 1919. Co. and Bn. Comdr., 11th Engrs.; Cambrai, St. Mihiel, Toul Sector, and Meuse-Argonne operations. Military Cross, Great Britain; Citation from Gen. Pershing for exceptionally meritorious service, Meuse-Argonne offensive.

DRUMMOND, WILLIAM WORRELL
Entered service Dec. 28, 1917; Capt., Engrs., N. A., Sept. 28, 1917. Overseas service July 10, 1918–May 23, 1919. Discharged Aug. 6, 1919. Sales Officer, London, England; attached to 23d Engrs. during St. Mihiel offensive. One star.

DRURY, WILLIAM FISHER
Entered service May 16, 1918; Capt., Engrs., N. A. Discharged Apr. 5, 1919. Co. Comdr., 1st Replacement Regt., Engrs.

DuBOIS, GEORGE BACHE
Entered service May 8, 1917; 1st Lt., E. O. R. C., Apr. 27, 1917. Overseas service, Dec. 8, 1917–Aug. 25, 1919. Discharged Sept. 26, 1919. C. O., 6th Engr. Train. Cited in Divisional Orders, 3d Div. Wounded Oct. 26, 1918, at Cunel, Meuse-Argonne. Six stars.

DUNLAP, WALTER HANNA
Entered service May 1, 1917; 1st Lt., E. O. R. C., Jan. 19, 1917; Capt., Engrs., U. S. A., Sept. 12, 1918. Discharged Dec. 12, 1918. With 306th Engrs. at Camp Jackson; 109th Engrs. at Camp Cody; 533d Engrs. at Camp Pike; with 3d and 4th Engr. Training Regts. at Camp Humphreys; with 212th Engrs. at Camps Forrest and Devens.

DUNN, BEVERLY CHARLES
Maj., Engrs., U. S. A.*

DUNSHEE, BERTRAM KELLOGG
Entered service May 8, 1917; 2d Lt., Engrs., N. A., Aug. 27, 1917; 1st Lt., Engrs., N. A., Dec. 29, 1917; Temporary Capt., Engrs., U. S. A., Sept. 3, 1918. Overseas service July 5, 1918–Feb. 14, 1919. Discharged Apr. 3, 1919. Topographic Officer, 316th Engrs. St. Mihiel and Meuse-Argonne offensives. Two stars. Wounded once.

DUPRE, WALLACE DUNCAN
Entered service Oct. 1, 1917, as 2d Lt., Inf., N. A.; 1st Lt., M. T. C., U. S. A., Sept., 1918. Overseas service Jan. 15, 1918–June 20, 1919. Discharged June 26, 1919. With 303d Repair Unit, in chg. gasoline and oil station at Nevers, France; Motor Transportation Despatcher, Nevers and Verneuil, France. Citation for meritorious service.

DURHAM, HENRY WELLES
Entered service Sept. 2, 1917; Capt., E. O. R. C., July 20, 1917; Maj., Engrs., N. A., Dec. 11, 1917. Overseas service Feb. 25, 1918–Oct. 1, 1919. Discharged Oct. 17, 1919. C. O., 41st Engr. Road Bn. in chg. forestry operations near St. Dizier, France; with Sec. Engr., Intermediate Sec., A. E. F., in chg. bldg. constr.; in chg. Road Sec., Div. Constr. and Forestry, Office, Chf. Engr., A. E. F., supervising all road work, Lines of Communication, A. E. F. Officier du Merite Agricole.

DURHAM, LEICESTER
Entered service May 13, 1917; Capt., E. O. R. C., June 28, 1917. Overseas service Feb. 26, 1918–Feb. 23, 1919. Discharged Feb. 28, 1919. Instr., E. O. T. S., Washington, D. C.; with 23d Engrs.; with 116th Engrs. as Topographic Officer and Personnel Adj.

DYKEMAN, CONRAD FRANCIS
1st Lt., Engrs., U. S. A., A. E. F.*

EAGER, VERNON MILTON
Entered service Nov. 9, 1917; 1st Lt., E. O. R. C., May 5, 1917; Capt., Q. M. C., Constr. Div., Mar. 18, 1918. Discharged Dec. 13, 1918. Asst. to Const. Q. M. at Raritan Arsenal; Asst. to Superv. Const. Q. M. on port terminals, Washington, D. C.

EASON, FRANK GARY
Entered service Jan. 1, 1918; Capt., Engrs., N. A., May 22, 1918. Overseas service June 16, 1918–Mar. 5, 1919. Discharged Apr. 5, 1919. Co. Comdr., 317th Engrs., during entire service. Meuse-Argonne offensive. Two stars.

EASTON, RUSSELL BURNS
Entered service July 30, 1917, as Capt., Engrs., N. A. Overseas service Jan. 7, 1918–Mar. 17, 1919. Discharged Mar. 21, 1919. Post Engr. at G. H. Q., A. E. F.; with War Damages Sec., U. S. Peace Comm.

EATON, HERBERT NELSON
Entered service Feb. 8, 1918, as Pvt., Sig. C., N. A.; Sgt., 1st Class, B. A. P., Dec. 15, 1918. Discharged Feb. 24, 1919. School of Aerial Photography. Cornell Univ.; 66th Aero Squadron, Eberts Field; Bureau of Standards, Washington, D. C.

ECKEL, EDWIN CLARENCE
Entered service May 1, 1917; Capt., E. O. R. C., Jan. 23, 1917; Maj., Engrs., U. S. A., Apr. 9, 1919. Overseas service Aug. 11, 1917–July 17, 1919. Discharged Aug. 9, 1919. Various staff assignments; C. O. of three companies, engr. troops.

EDDY, ALBERT CLARK
Entered service June 1, 1918, as Capt., Engrs., N. A. Overseas service June 30, 1918–July 4, 1919. Discharged July 26, 1919. Technical Officer, Montierchaume project; Engr. Officer in chg. constr. at Poitiers; Dist. Engr. of Roads.

EDDY, HAROLD MANSFIELD
Entered service Mar. 13, 1918, as Lt., Jr. Grade, C. E. C., U. S. N. R. F.; Lt., Aug. 29, 1918; Lt. Comdr., July 6, 1919. Asst. to, and Officer in Chg., constr. Navy and War Bldgs., Washington, D. C.; in chg. constr. dirigible hangar at Lakehurst, N. J.

EDGERTON, GLEN EDGAR
Entered service June 23, 1904; through all grades in C. of E., U. S. A., to Maj., May 15, 1917; Lt. Col., Aug. 5, 1917; Col., July 13, 1918. Returned to permanent grade of Maj., Mar. 21, 1920. Organized and trained 2d, 5th, 209th, 210th, 211th, 212th, 213th and 214th Engrs.

EDMONDSON, RALPH SELDEN
1st Lt., Engrs., U. S. A.*

EDMONSTON, ARTHUR DONALD
2d Lt., Engrs., U. S. A.*

EDWARDS, GEORGE GARRETT
Entered service May 8, 1917; Capt., Engrs., N. A., Aug. 15, 1917. Overseas service June 13, 1918–June 15, 1919. Discharged July 10, 1919. Co. Comdr., 315th Engrs.; St. Mihiel and Meuse-Argonne offensives; Sazerais Sector; Army of Occupation, Germany. Three stars.

EDWARDS, LATTA VANDERION
Entered service Oct. 16, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 19, 1918. With 138th Engrs.

EDWARDS, LLEWELLYN NATHANIEL
Capt., Engrs., U. S. A.*

EDWARDS, WILLIAM WATKYN
Entered service July 5, 1918, as Pvt., C. A. C., N. A.; 1st Lt., C. A. C., U. S. A., Dec. 19, 1918. Discharged Dec. 23, 1918. With 9th Co., C. A. C., Oahu, Honolulu; C. A. school troops.

ELDREDGE, CHARLES GUY
Entered service Jan. 3, 1918, as 1st Lt., San. C., N. A. Overseas service July 18, 1918–May 12, 1919. Discharged May 28, 1919. C. O., San. Squad Nos. 2 and 56, Water Supply; water supply and purification plant constr. and operation at St. Nazaire, France.

ELLIOTT, JAMES WILLIAM
Capt., Engrs., U. S. A., A. E. F.*

ELLIOTT, MALCOLM
Entered service May 15, 1917, as Capt., E. O. R. C.; Maj., E. O. R. C., Aug. 15, 1917; Lt. Col., Engr. R. C., June 5, 1918. Overseas service Sept. 9, 1918–June 30, 1919. Discharged July 28, 1919. Bn. Comdr., 2d in command, and C. O., 309th Engrs.; C. O., 310th and 602d Provisional Regts.

ELLIOTT, PERCIVAL
Entered service Apr. 4, 1918, as Pvt., Engrs., N. A.; Cpl., Engrs., U. S. A., Feb. 5, 1919. Overseas service May 10, 1918–May 23, 1919. Discharged June 12, 1919. With 33d Engrs. in gen. constr.

ELLIS, GWYNNE WALLACE
Entered service May 5, 1918, as Capt., Engrs., N. A. Discharged Feb. 28, 1919. With 1st Replacement Regt., Washington Barracks; with 210th Engrs.

ELLISON, ALEXANDER HALL
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., June 19, 1917; Capt., Engrs., N. A., Aug. 3, 1918. Overseas service Jan. 4, 1918–Jan. 3, 1919. Discharged Jan. 15, 1919. With 20th Engrs. at Camp Belvoir, topographic mapping; Office, Chf. Engr., A. E. F., assigned to Forestry Sec.; Asst. Officer in chg. Gen. Engr. Depot, A. E. F.; Office, Chf. Engr., A. E. F., in chg. Forestry Sec., and asst. in chg. Gen. Engr. Depot. One star.

ELTINGE, ORVILLE LAMONT
Entered service Aug. 17, 1918; 1st Lt., Engrs., U. S. A., Aug. 10, 1918. Discharged Jan. 15, 1919. Various duties at Camp Humphreys.

ELY, CARL BRANDES
Maj., A. S., U. S. A.*

EMBURY, AYMAR, 2d
Entered service Sept. 8, 1917, as Capt., Engrs., N. A.; Maj., Engrs. Overseas service Apr. 28, 1918–May 24, 1919. Discharged May 28, 1919. Office, Chf. of Engrs., Washington, D. C.; with 40th and 305th Engrs.; General Staff, G. H. Q., A. E. F.; Champagne, Marne, Marne-Aisne, Oise-Aisne, St. Mihiel, Meuse-Argonne operations; Toul and Vesle Sectors. Citation from Gen. Pershing. Five stars.

EMERSON, GEORGE DANA
Entered service Oct. 15, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Feb. 20, 1919. Asst. Const. Q. M., Army Supply Base, Boston, Mass

EMERSON, RAFFE
Lt., Flying Corps, U. S. N. R. F.*

EMORY, LLOYD TILGHMAN
Entered service Sept. 27, 1918; Capt., Ord. Dept., U. S. A., Nov. 5, 1918. Discharged Dec. 16, 1918. Progress Sec., Estimates and Requirements Div., Office, Chf. of Ord., Washington, D. C.

ENDICOTT, MORDECAI THOMAS
Entered service July 14, 1874; Comdr., Capt., Commodore and Rear-Admiral, C. E. C., U. S. N. Pres., Examining Bd. for entrance into reserve and regular service, C. E. C., U. S. N., Bureau Yards and Docks, Washington, D. C.; Member, Naval Bd. on Award of Medals.

ENGER, ARTHUR LUDWIG
Entered service June 21, 1917; Capt., E. O. R. C., May 5, 1917. Discharged Sept. 27, 1919. Officer in chg. Engr. Sub-Depot, Camp Cody; Zone Supply Officer, El Paso, Tex.

ESTES, FRANKLIN EDWARD
Entered service May 14, 1917; Capt., E. O. R. C., May 10, 1917; Maj., Army Service Corps., U. S. A., Oct. 15, 1918; Lt. Col., Army Service Corps, U. S. A., Feb. 17, 1919. Overseas service July 28, 1917–Oct. 28, 1919. Discharged Oct. 30, 1919. Co. Comdr., Bn. Adj. and Supply Officer, 17th Engrs.; in chg. Administrative Labor Cos.; Director, Army Service Corps. Citation from Gen. Pershing for meritorious service; Chevalier, Legion d'Honneur; Officer, Order of the Crown, Italy.

EVANS, EDWARD MONTAGUE
Entered service Oct. 28, 1918, as 1st Lt., M. T. C., U. S. A. Discharged Oct. 16, 1919. Office of Chf., M. T. C., Washington, D. C., in chg. convoy service of Operations Div.

EVERETT, CHESTER McKENZIE
Entered service Aug. 29, 1918, as Capt., San. C., U. S. A. Discharged Dec. 12, 1918. Camp San. Engr., Camp Funston.

EVERETT, RALPH BURROWS
Entered service Nov. 21, 1917, as Pvt., Engrs., N. A.; Pvt. 1st Class, Dec., 1917. Overseas service Mar. 30, 1918–Mar. 5, 1919. Discharged July 19, 1919. With 23d Engrs. in gen. constr., mainly highway work.

EVERHAM, ARTHUR CASSIDY
Entered service Apr. 30, 1918; Maj., Q. M. C., Constr. Div., N. A., Apr. 30, 1918. Discharged Feb. 16, 1919. Asst. Sec. Chf., Ord. Group, Constr. Div.

FAHY, JOSEPH AUGUSTINE
Entered service Dec. 8, 1917; 1st Lt., Engr. R. C., Aug. 30, 1917; Capt., Engrs., N. A., June 17, 1918. Overseas service, Sept. 3, 1918–Aug. 5, 1919. Discharged Aug. 29, 1919. F. A. O. T. C., Fort Oglethorpe; with 307th Engrs., Camp Gordon; Co. Comdr., 518th Engrs., Camp Gordon and in France; H. Q., 2d G. Div., T. C., as Adj., 2d G. Div., T. C., and Camp Ancona (Bassens), France.

FAIRCHILD, JOHN FLETCHER
Entered service Jan. 28, 1918; Maj., Engrs., N. A., Feb. 1, 1918. Overseas service Sept. 4, 1918–June 25, 1919. Discharged July 25, 1919. H. Q. Brig., 1st Corps Troops, Camp Wadsworth; in chg. sewers, road and bridge work, 3d Bn., 56th Pioneer Inf.; Asst. Comdr., Sub-Post at West Trier, Germany, 3d Army; Meuse-Argonne offensive. One star.

FAIRLIE, JOHN WALKER
1st Canadian Expeditionary Force, B. E. F.*

FAIRMAN, JAMES ROBERT
Capt., T. C., U. S. A., A. E. F.*

FALK, MYRON SAMUEL
Entered service Dec., 1917, as Maj., Ord. C., N. A.; Lt. Col., Ord. C., Apr., 1919. Discharged July, 1919.

FARLEY, MARCUS MARTIN
Entered service Sept. 7, 1917; 1st Lt., Engrs., N. A., Nov. 9, 1917. Discharged Dec. 18, 1917. With 143d Engrs.

FARRIN, JAMES MOORE
Entered service Sept. 2, 1917, as Capt., Engrs., N. A. Overseas service Jan. 24, 1918–Aug. 1, 1919. Discharged Jan. 23, 1920. With 20th, 21st, 302d and 2d Engrs.; Adj., 6th Bn., Special Forestry Regt.

FASSETT, EARLE WALKER
Capt., Engrs., U. S. A., A. E. F.*

FAWCETT, PHILIP NORRISON
Entered service Mar. 18, 1916, as 2d Lt., Royal Engrs., British Army; 1st Lt., Sept. 18, 1917; Capt., Aug., 1919. Overseas service Feb. 1, 1916–Aug. 28, 1919. Discharged Aug. 20, 1919. With 232d Light Ry. Forward Co.

FEIGEL, JOHN HENRY
Entered service Aug. 9, 1917; 1st Lt., E. O. R. C., June 16, 1917. Overseas service Jan. 15, 1918–June 6, 1919. Discharged Aug. 15, 1919. With 1st and 305th Engrs.; Bn. Supply Officer, 20th Engrs. Citation for especial service and bravery, Meuse-Argonne offensive. Three stars. One wound.

FEINER, MARK ANTONY
Entered service Apr. 20, 1918, as Pvt., Sig. C., N. A.; Sgt., Sig. C., U. S. A., Dec. 20, 1918. Discharged Apr. 21, 1919. With Meteorological Sec.

FELLER, FRANK HENRY
Entered service Oct. 18, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 4, 1918. Camp Humphreys.

FELTHAM, PERCY MARSHALL
Entered service June 7, 1918, as Capt., Engrs., N. A. Discharged Mar. 30, 1920. With 79th Engrs.; wounded by premature explosion of hand grenade.

FENSTERMAKER, DEWITT CLINTON
Capt., Engrs., U. S. A.*

FERGUSON, HARRY FOSTER
Entered service May 13, 1917; 1st Lt., E. O. R. C., June 15, 1917. Overseas service Dec. 11, 1917–Jan. 25, 1919. Discharged Jan. 31, 1919. With H. Q., Advance Sec., A. E. F.

FERNALD, GORDON HILDRETH
Entered service May 12, 1917; 1st Lt., E. O. R. C., Feb. 14, 1917; Capt., Engr. R. C., Apr. 15, 1918. Overseas service July 7, 1918–May 29, 1919. Discharged June 7, 1919. With 304th Engrs. Divisional Citation. Two stars. One wound.

FICKES, CLARK ROBINSON
Entered service July 16, 1918, as Maj., Engrs., N. A. Overseas service Aug. 15, 1918–Jan. 20, 1919. Discharged Jan. 30, 1919. C. O. in chg. constr., Talmont project, France.

FIEBEGER, GUSTAVE JOSEPH
Entered service June 13, 1879; through all grades in C. of E., U. S. A., to Capt., 1896, when transferred to U. S. Military Academy as Prof. of Eng. Overseas service Oct. 1917–Apr., 1919. Military Observer, British and French fronts and schools of British, French and American Armies.

FIELD, ARTHUR MAXWELL
Entered service June 21, 1918, as Pvt., C. A. C., N. A.; 2d Lt., C. A. C., U. S. A., Sept. 25, 1918. Discharged Dec. 13, 1918. Advance Orientation School for Officers; Bn. Orientation Officer, 40th Artillery.

FIFER, FRANK PRESTON
Entered service May 6, 1917; Capt., E. O. R. C., June 13, 1917. Overseas service Aug. 9, 1917–Mar. 1, 1919. Discharged Jan 13, 1920. Supt. of constr., base hospitals, A. E. F., with French labor.

FINLEY, CHARLES MacFARLANE
Capt., Engrs., U. S. A., A. E. F.*

FINNELL, WOOLSEY
Entered service Sept. 12, 1917; Maj., E. O. R. C., June 19, 1917; Lt. Col., Engrs., U. S. A., Sept. 14, 1918. Overseas service Nov. 26, 1917–July 5, 1919. Discharged Sept. 16, 1919. Bn. Comdr., 20th Engrs.; C. O. 501st Engrs. and U. S. troops at A. P. O. 741, A. E. F.; with U. S. Peace Commission in charge transportation and public utilities; G-5, G. H. Q., Paris Section, in chg. field work of Dept. of Citizenship. Legion d'Honneur; Citation from Gen. Pershing for meritorious service.

FISHER, GEORGE JOSEPH
Entered service Oct. 12, 1917, as Pvt., Engrs., N. A.; 2d. Lt., Engrs., N. A., Aug. 1, 1918; 1st Lt., Engrs., U. S. A., Oct. 1, 1918. Discharged Feb. 6, 1919. With 316th and 219th Engrs., and 219th Engr. Train.

FISHER, JANON
Entered service Dec. 28, 1917; Maj., E. O. R. C., July 10, 1918. Overseas service Aug. 29, 1918–Apr. 25, 1919. Discharged Apr. 29, 1919. C. O., 522d Engrs., Camp Humphreys and with 1st and 2d Armies, A. E. F., in Meuse-Argonne offensive. One star.

FISK, CLINTON HINCKLEY
Entered service Apr. 5, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged July 10, 1919. Superv. Const. Q. M., Army Supply Bases at Boston, Brooklyn and New Orleans.

FISKE, HAROLD LA SALLE
Entered service 1917 as Pvt., N. A.; 2d Lt., Inf., N. A., 1917. Overseas service, 1918. O. T. C., Camp Upton; with 77th Div., A. E. F.; student and instr., Machine Gun School, A. E. F.; with 305th Machine Gun Bn. Killed Oct. 6, 1918, in action in Argonne Forest, France.

FITZ GERALD, CHRISTOPHER COLUMBUS
Entered service Jan. 12, 1918; Maj., Engrs., N. A., Sept. 19, 1917. Overseas service June 15, 1918–June 9, 1919. Discharged July 29, 1919. With 32d Engrs.; training engr. units for overseas duty; in chg. St. Sulpice supply base project, Gironde, France; Supt. Roads, Base Sec. No. 2, A. E. F.

FITZ GERALD, GERALD CHAPMAN
Entered service June 19, 1917, as 1st Lt., E. O. R. C.; Capt., Engrs., U. S. A., Sept. 18, 1918. Overseas service Sept. 24, 1918–Aug. 27, 1919. Released from active duty Oct. 1, 1919. With 7th and 319th Engrs.

FITZPATRICK, FRANCIS JAMES
Entered service Feb. 23, 1917, as Capt., E. O. R. C. Overseas service Jan. 23, 1918–June 6, 1919. Discharged July 23, 1919. With 316th Engr. Train and 508th Engrs. as Co. Comdr.; Asst., Roads Officer, 1st Army, and Dist. Roads Officer, Base Sec. No. 7, A. E. F.; Seicheprey and Xivray operations in Toul Sector; St. Mihiel and Meuse-Argonne offensives. Three stars.

FITZPATRICK, MARK WILLIAM
Sgt., Engrs., U. S. A., A. E. F.*

FLAGG, HERBERT JUDSON
Entered service July 25, 1917, as 1st Lt., C. A. C., N. G.; Capt. C. A. C., N. A., Nov. 27, 1917. Discharged May 7, 1919. Adj., Ft. Flagler, Wash.; Instr., C. A. School, Ft. Monroe, Va.; Coast Defense Artillery Engr.

FLAGLER, CLEMENT ALEXANDER FINLEY
Entered service June 10, 1885; through all grades in C. of E., U. S. A., to Brig. Gen., Dec. 17, 1917; Maj. Gen., Engrs., U. S. A., Oct. 1, 1918. Overseas service May 26, 1918–Apr. 7, 1919. C. O., 7th Engrs.; C. O., 5th F. A. Brig.; C. O., Corps Artillery, 3d Corps; C. O., 42d Div., St. Mihiel and Meuse-Argonne offensives. Officier, Legion d'Honneur; Croix de Guerre with Palm. Three stars.

FLEMING, BURTON PERCIVAL
Entered service May 7, 1918; Capt., Engrs., N. A., Jan. 28, 1918. Overseas service Aug. 15, 1918–July 27, 1919. Discharged Aug. 15, 1919. With 4th Bn., 34th Engrs.; Executive and Shop Officer, Advance Engr. Supply Depot No. 1, A. E. F.

FLICK, JOHN KRAMER
Entered service Dec. 17, 1917; Capt., E. O. R. C., June 23, 1917. Discharged Mar. 7, 1919. Asst. to Const. Q. M., Ord. Depot, Curtis Bay, Md.

FLOOK, LYMAN RUSSELL
Entered service Jan. 3, 1918; 1st Lt., Ord. C., Nitrate Div., N. A., Dec. 14, 1917. Discharged May 31, 1919. Insp., U. S. Nitrate Plant No. 1, in chg. constr.

FOGERTY, MERTON SHUMWAY
Entered service Apr. 17, 1918, as Pvt., Aviation Sec., Sig. C., N. A.; Sgt., Sig. C., U. S. A., Nov. 1, 1918; Sgt., 1st Class, Sig. C., U. S. A., Feb. 1, 1919. Overseas service Aug. 31, 1918–June 16, 1919. Discharged June 23, 1919. H. Q., Meteorological Sec., Sig. C., A. E. F., as Map Maker and Weather Forecaster; H. Q., 3d Army, A. E. F., in chg. map room.

FOGG, ALDEN KNOWLTON
Entered service June 15, 1917, as Lt., Jr. Grade, C. E. C., U. S. N.; Lt., Oct. 15, 1917. On duty at U. S. Naval Academy; Asst. Public Works Officer, U. S. Navy Yard, Portsmouth, N. H.

FOLLIN, JAMES WIGHTMAN
Entered service Nov. 9, 1917, as 1st Lt., San. C., N. A.; Capt., San. C., U. S. A., Aug. 23, 1918. Discharged Oct. 16, 1919. H. Q., 35th Div., Ft. Sill, Okla.; in chg. water supply; Surgeon General's Office, Washington, D. C.

FOLLING, BJORNE NICOLAS
Capt., Engrs., U. S. A., A. E. F.*

FOOTE, OLNEY NORMAN
Entered service Feb., 1917, as Sgt., Cavalry, 1st N. Y.; 2d Lt., Cavalry, N. A., Sept., 1917; 1st Lt., Inf., U. S. A., Feb., 1919. Overseas service July, 1918–June, 1919. Discharged June 30, 1919. In cavalry attached to 110th Inf.; with 322d Inf.

FORBES, FRANCIS BONNER
Entered service Aug. 7, 1918; 1st Lt., Engrs., N. A., July 13, 1918. Discharged Dec. 19, 1918. With 22d Engrs. at Ft. Benjamin Harrison, Ind.; with 55th Engrs., at Ft. Leavenworth, Kans.

FORBES, HYDE
Entered service June, 1918, as Pvt., Engrs., N. A.; Cpl., Engrs., N. A., Aug., 1918; Sgt., Engrs., U. S. A., Sept., 1918; 1st Lt., Engrs., U. S. A., Oct. 1, 1918. With 1st Replacement Regt.; Instr., E. O. T. S.

FORD, WILLIAM ELLIS
Entered service July, 1917; Capt., E. O. R. C., May 15, 1917. Discharged Oct. 2, 1919. Asst. Const. Q. M., Camp Pike and St. Louis Warehouse; Const. Q. M. at Camp MacArthur.

FORSTER, ARTHUR OSCAR
Capt., Q. M. C., U. S. A.*

FORTER, CECIL ALFRED
Entered service Aug. 26, 1918, as 1st Lt., Sig. C., U. S. A. Discharged Jan. 30, 1919. Asst. Camp Engr., and Camp Engr., Camp Greenleaf.

FORTER, SAMUEL ALEXANDER
Entered service Apr. 11, 1918, as Lt. Jr. Grade, C. E. C., U. S. N. R. F.; Lt., Nov. 1, 1918. Overseas service May 20, 1918–June 10, 1919. Released from active service July 19, 1919. Public Works Officer, Pauillac, and Aide for Aviation, Brest, France. Gold medal from City of Lille, Nord, France.

FORTNEY, CAMDEN PAGE
Entered service Sept. 2, 1917, as Capt., Engrs., N. A. Discharged Feb. 6, 1919. American Univ.; Gen. Engr. Depot.

FOSTER, CLARENCE MARVIN
Entered service Jan. 6, 1918; Capt., Q. M. C., Constr. Div., N. A., Apr. 4, 1918; Maj., Q. M. C., Constr. Div., U. S. A., Apr. 29, 1919. Contr. Officer, Procurement Div.; Member, Bd. of Cont. Review, and Claims Bd., Constr. Div.

FOSTER, ERNEST HOWARD
Entered service Mar. 29, 1917, as Ensign, U. S. N. R. F. Released from active duty Nov. 18, 1917. Executive Officer, Sec. Base No. 8, Coast Patrol.

FOSTER, SAMUEL DAVIS
Entered service Nov. 29, 1917, as 1st Lt., F. A., N. A.; Capt., F. A., U. S. A., Sept. 4, 1918. Overseas service Apr. 30, 1918–Apr. 5, 1919. Discharged Apr. 8, 1919. Telephone Officer and Munition Officer, Brig. Staff, 53d F. A., Divisional citation, 91st Div.; Belgian War Cross. Gassed at Fismes.

FOUILHOUX, JACQUES ANDRE
Entered service May 15, 1917; Capt., F. A., N. A., Aug. 15, 1917. Overseas service May 20, 1918, to Aug. 15, 1919. Discharged Sept. 3, 1919. School of Fire, F. A., Ft. Sill; Instructor F. A. Training Camp, Camp Doniphan; Operation Officer, 129th F. A., Langres, France; Conducting Officer, G-2-E, G. H. Q., A. E. F.; Liaison Officer, G-5, G. H. Q., A. E. F. Three stars.

FOUNTAIN, THOMAS LILLY
Entered service Nov. 16, 1917; Capt., E. O. R. C., June 23, 1917. Overseas service Dec. 12, 1917–Aug. 12, 1919. Discharged Aug. 29, 1919. With Chf. Engr., Base Section No. 1, A. E. F., in sewer design and constr., St. Nazaire, France.

FOWLER, CHARLES WORTHINGTON
Entered service Sept. 14, 1917; 1st Lt., A. S., N. A., Sept. 4, 1917. Overseas service Mar. 4, 1918–May 1, 1919. Discharged June 7, 1919. C. O., 73d, later 485th, Aero Constr. Squadron.

FOWLER, FRANK GEORGE
Entered service Nov. 5, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Jan. 9, 1919. In chg. road constr., Yorktown to Newport News, Va.

FOWLER, FREDERICK HALL
Entered service July 25, 1918; Capt., Engrs., U. S. A., Aug. 9, 1918. Discharged Dec. 23, 1918. Office, Chf. of Engrs., Washington, D. C.; Topographic Officer, 211th Engrs.

FOX, HENRY
Entered service Sept. 1, 1917; Capt., E. O. R. C., June 17, 1917. Discharged Sept. 22, 1918. Army Service School, Ft. Leavenworth; with 314th Engrs., Camp Funston; with 312th Engrs., Camp Pike; with 543d Engrs., Camp Pike; with 12th Engrs. Replacement Regt.

FOX, STEPHENSON WATERS
Entered service Nov. 22, 1917; Maj., Engrs., N. A., Nov. 3, 1917. Discharged Mar. 31, 1919. Asst. to Secy., Inland Water Transportation Comm., Office, Chf. of Engrs.; Engr. of Contracts and Constr., Div. of Inland Waterways, U. S. R. R. Administration.

FRANK, GEORGE STEDMAN
Entered service July 17, 1917, as 1st Lt., Aviation Sec., S. C., U. S. R.; Capt., A. S., U. S. A., Oct. 3, 1918. Overseas service July 18, 1917–Oct. 19, 1919. Discharged Oct. 21, 1919. Asst. Chf. Constr. Officer, Aerial Gunnery School, St. Jean des Monts, France; Chf. Constr. Officer, 2d and 3d Aviation Instruction Centers, Tours and Issoudun, respectively; Chf., Expeditions Sec., American Relief Administration, Paris.

FRANKLIN, WILLIAM HAWLEY
Entered service Sept. 10, 1918, as 1st Lt., Q. M. C., Constr. Div., U. S. A. Discharged Jan. 20, 1919. Asst. to Officer in Chg. Constr., Bethlehem Loading Co's Plant, Mays Landing, N. J.

FRANKS, JOHN BRANDON
Entered service Sept. 18, 1917, as 1st Sgt., Inf., N. A.; 2d Lt., F. A., N. A., June 1, 1918; 1st Lt., F. A., U. S. A., Sept. 22, 1918. With 353d Inf.

FREEMAN, WILLIAM BRADLY
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 23, 1917; Maj., Engrs., U. S. A., Apr. 7, 1919. Overseas service Feb. 19, 1918–June 14, 1919. Discharged July 20, 1919. Co. and Bn. Comdr., 509th Engrs.; Engr. in chg. constr., St. Nazaire and Montoir filter plants, France. Citation from Gen. Pershing for meritorious service.

FRENCH, FRANK CHAUNCEY
Entered service Dec. 26, 1917; Capt., E. O. R. C., July 10, 1917; Maj., Engrs., N. A., July 30, 1918. Overseas service July 9, 1918–Sept. 10, 1918. Discharged Jan. 8, 1919. Co. Comdr., 525th Engrs., in varied constr. work in U. S. and France; trained 79th Engrs., Camp Leach, for sapper duty.

FRICKSTAD, WALTER NETTLETON
Entered service Oct. 19, 1918, as Capt., Engrs., U. S. A. Discharged Feb. 1, 1919. E. O. T. S., Camps Douglas and Humphreys.

FRIEDMAN, HARRY BAYARD
Entered service Aug. 23, 1917; 2d Lt., A. S., N. A., Nov. 27, 1917; 1st Lt., A. S., N. A., Apr. 1, 1918. Discharged Jan. 6, 1919. Asst. Engr. Officer, Ellington Field; C. O., 286th Aero Squadron and recruit detachments; Officer in chg. Inf. training; flying duty at Kelly Field.

FRIES, AMOS ALFRED
Entered service June 15, 1894; through all grades in C. of E., U. S. A., to Lt. Col., May 15, 1917; Col., Aug. 5, 1917; Brig. Gen., U. S. A., Aug. 16, 1918. Overseas service July 23, 1917–Dec. 18, 1918. Reverted to rank Lt. Col., Feb. 7, 1919. Director of Roads; Chf. of Gas Service, C. W. S.; C. O., Edgewood Arsenal. Distinguished Service Medal; Commandeur, Legion d'Honneur; Companion, Order of St. Michael and St. George, Great Britain; Special citation from G. H. Q., A. E. F. Three stars.

FRIESELL, FRANK McCLAREN
Entered service May 13, 1917; Capt., Engrs., N. A., July 20, 1917, unassigned. Discharged Nov. 15, 1917. Re-enlisted Feb. 12, 1918, as 1st Sgt., Engrs, N. A. Discharged Dec. 21, 1918. With 319th Engrs.; F. A. Officers' Training School.

FROISETH, RICHARD EUGENE
1st Lt., Engrs., U. S. A.*

FROST, EDWARD MURRAY
Lt., Jr. Grade, C. E. C., U. S. N.*

FRY, ALFRED BROOKS
Entered service May 25, 1886; through all grades to Comdr., U. S. N., Mar. 4, 1910; Capt., U. S. N., Mar. 27, 1916; Capt., U. S. N. R., July 1, 1918. Overseas service Dec. 7, 1917–Mar. 1, 1918. Engr. observation duty, U. S. S. *Leviathan;* Engr. Aide to Rear Admiral Burd; Ind. Mgr., Navy Yards, Brooklyn, N. Y., and 3d Naval Dist.; served at Paris, London, Liverpool, Calais, Brest, etc.

FURLOW, FELDER
Entered service Sept. 25, 1918; Capt., Engrs., U. S. A., Nov. 20, 1918. Discharged Dec. 18, 1918. Co. Comdr., 120th Engrs., Ft. Benjamin Harrison.

FYFE, JAMES LINCOLN
Entered service Mar. 23, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged Oct. 17 1919. Eng. Branch, Constr. Div., at Washington, D. C.; Constr. Q. M. of the Little Rock Ry. and Electric Light Plant, Little Rock, Ark.; Constr. Q. M., Camp Pike.

GABELMAN, CHARLES GROVER
Entered service July 29, 1918, as 1st Lt., Engrs., N. A. Discharged June 4, 1919. With 2d Engr. Training Regt., Camp Humphreys.

GAILOR, CHESTER FRANCIS
Entered service Apr. 26, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged Apr. 3, 1919. Chf., Industrial Service Sec., in chg. labor conditions, wage rates, etc.

GALBREATH, ALBERT WEBSTER
Entered service May 13, 1917; 1st Lt., E. O. R. C., June 16, 1917; Capt., Engrs. N. A., Mar. 27, 1918. Overseas service July 28, 1917–Apr. 27, 1919. Discharged May 28, 1919. With 12th Engrs.; Maintenance, Asst. Traffic Officer and Traffic Officer, Somme Sector, Light Rys.; Asst. Supt., and Supt., Light Rys., St. Mihiel Sector. Four stars.

GALLAGHER, LEONARD BRUCE
Entered service June 19, 1917, as 2d Lt., E. O. R. C.; 1st Lt., and Capt., Engrs. U. S. A., July 7, 1918. Overseas service Jan. 1, 1918–July 22, 1919. With 117th, 116th, and 301st Engrs.

GALLOWAY, JOHN DEBO
Entered service Nov. 7, 1917; Maj., E. O. R. C., May 5, 1917. Overseas service Dec. 11, 1917–Jan. 22, 1919. Discharged Jan. 24, 1919. Gen. Staff, G. H. Q., A. E. F., assigned to G-2 (Intelligence). Citation from Gen. Pershing for meritorious service.

GALVIN, JAMES AUGUSTINE
Entered service May 8, 1917; 2d Lt., E. O. R. C., Feb. 22, 1917; 1st Lt., Engrs., N. A., July 20, 1918. Overseas service Dec. 11, 1917–May 1, 1919. Discharged Oct. 31, 1919. With 301st Engrs. at Camp Devens; Engr. Officer, Gen. Staff College, Langres, France; Asst. Camp Engr., Camp Cox. Citation from Brig. Gen. W. A. Smith for ability and efficiency. Three stars.

GARDNER, WARREN
1st Lt., Engrs., U. S. A.*

GARLINGHOUSE, RALPH LEMAN
Entered service July, 1918, as Chf. Machinist's Mate, U. S. N. R. F.; Warrant Machinist, Jan., 1919; Ensign, May, 1919. Released from active duty June 16, 1919. U. S. S. *Wassaic.*

GARNETT, BENJAMIN JAY
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., June, 1917. Overseas service Jan. 26, 1918–Mar. 1, 1919. Discharged Mar. 5, 1919. Constr., Advance Sec., S. O. S., A. E. F.

GARRETT, ROBERT PEEL
Entered service July 16, 1918, as Capt., Ord. C., N. A. Discharged July 9, 1919. On duty with Ry. and Seacoast Carriage Sec., Arty. Div., Ord. Dept., Washington, D. C.

GATENS, RAY STUART
Entered service Mar. 1, 1918, as 1st Lt., Engrs., N. A.; Capt., Engrs., U. S. A., Jan. 26, 1919. Overseas service May 10, 1918–May 24, 1919. Discharged June 23, 1919. With 42d, 43d and 103d Engrs.; 1st Army, A. E. F.

GAUMER, ALBERT WESLEY
Entered service Aug. 12, 1918; 1st Lt., Engrs., U. S. A., Nov. 14, 1918. Discharged Jan. 17, 1919. With 4th and 8th Engr. Training Regts; Co. Comdr., 4th Engr. Training Regt.

GAUSMANN, ROY WARNER
Entered service May 8, 1917; Capt., E. O. R. C., Feb. 23, 1917; Maj., Engr. R. C., Jan. 7, 1918. Overseas service May 8, 1918–Jan. 8, 1919. Discharged Jan. 11, 1919. Co. Comdr., and Bn. Comdr., 303d Engrs.; British front, Strazelle Sector; St. Mihiel and Meuse-Argonne offensives. Three stars. Wounded July 15, 1918.

GAYLER, ERNEST ROTTECK
Entered service Oct., 1902; through all grades in C. E. C., U. S. N., to present grade of Comdr.

GEIB, GEORGE ALBERT
Entered service May 11, 1917; 1st Lt., E. O. R. C., June 22, 1917; Capt., E. O. R. C., Aug. 15, 1917. Overseas service June 22, 1918–July 1, 1919. Discharged July 24, 1919. Adj., and Topographic Officer, 313th Engrs.; Co. Comdr., 527th Engrs.; St. Mihiel and Meuse-Argonne offensives. Two stars.

GENUNG, JAMES HOLCOMBE, JR.
Entered service May 26, 1917, as 2d Lt., E. O. R. C.; 2d Lt., F. A., N. A., Aug. 8, 1917; 1st Lt., F. A., N. A., Nov. 15, 1917; Capt., F. A., N. A., July, 1918; Maj., F. A., U. S. A., Sept., 1918. Overseas service May 1, 1918–Sept. 10, 1918. Co. Comdr., and Bn. Comdr., 21st F. A.; Bn. Comdr. 69th F. A.; C. O. 2d and 71st F. A.; Military Observer, 21st French Army Corps, Verdun and Champagne. Two stars.

GEORGE, HOWARD HOWELL
Entered service May 8, 1917; 1st Lt., E. O. R. C., Jan. 16, 1917; Capt., E. O. R. C., Aug. 15, 1917. Overseas service June 30, 1918–Jan. 26, 1919. Discharged Jan. 29, 1919. Co. Comdr., 305th and 55th Engrs.; Engr. Officer in chg. bldg. constr., Chateaureux Storage Depot Project; with Sec. Engr., Base Sec. No. 4, A. E. F.

GERNER, ANSON JOHN
Pvt., E. O. T. S., U. S. A.*

GETTY, LORENZO TODD
Entered service Nov. 2, 1917; 1st Lt., Aviation Sec., Sig. C., N. A., Oct. 25, 1917. Capt., A. S., U. S. A., Oct. 15, 1918; Maj., A. S., Feb. 21, 1919. Overseas service, Dec. 3, 1917–Apr. 20, 1919. Discharged May 7, 1919. Adj., 75th Aero Squadron; Constr. Supply Officer, Asst. Adj., Adj., and Executive Officer, A. S. Production Center No. 2. Chevalier, Ordre de l'Etoile Noire.

GIBBS, ELBERT ALLAN
Entered service Apr. 25, 1917; Capt., E. O. R. C., May 19, 1917; Maj., E. O. R. C., July 29, 1917; Lt. Col., Engrs., N. A., June 17, 1918; Col., Engrs., U. S. A., Feb. 13, 1919. Overseas service July 9, 1917–July 7, 1919. Discharged July 24, 1919. With 5th, 15th, 32d, and 311th Engrs.; H. Q., S. O. S., A. E. F.; Chf., Gen. Constr. Sec., Div. of Constr. and Forestry, A. E. F.; served on various boards. Citation from Gen. Pershing for meritorious service; Legion d'Honneur; Distinguished Service Medal. One star.

GIBBS, WILLIAM WETMORE†
1st Lt., Engrs., U. S. A., A. E. F.*

GIBSON, THOMAS FENNER
Entered service Sept. 3, 1918, as Pvt., Inf., U. S. A.; Cpl., Inf., U. S. A., Nov. 1, 1918. Discharged Jan. 30, 1919. With 63d Inf.

GIESEN, WALTER EDWARD
Entered service July 23, 1918, as Pvt., U. S. M. C. Discharged Jan. 23, 1919. With 15th Marines.

GIESTING, FRANK ALEXANDER
Entered service May, 1917, as Maj., E. O. R. C.; Lt. Col., Engrs., N. A., Aug., 1918; Col., Engrs., U. S. A., Nov. 15, 1918. Overseas service Mar. 1917–May, 1919. Discharged May, 1919. With 302d Engrs. Croix de Guerre.

GIFFELS, WILLIAM CHARLES
Entered service May 14, 1917; 1st Lt., Engrs., N. A., Aug. 26, 1917. Overseas service July 16, 1918–July 18, 1919. Discharged Aug. 6, 1919. With 310th Engrs. in North Russia Military Cross, Great Britain; Order of St. Anne, Russia.

GILES, JOHN ANGUS
Entered service Sept. 9, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Jan. 31, 1919. Asst. Officer in Chg. Utilities, Camp Upton; Officer in Chg. Utilities, Gen. Hosp. No. 2; Asst. to Officer in Chg., Port Utilities, New York City.

† Died Jan. 13, 1919.

GILMAN, EDGAR DOW
Entered service Mar. 15, 1918, as 1st Lt., C. W. S., N. A. Discharged Apr. 30, 1919. With 473d Engrs.; Chf. Gas Officer, Camp Greene; member Bd. of Officers for Determining Land Damage Claims; Camp Exchange Officer; Camp Adj.

GLADDING, JAMES NICKERSON
Entered service Sept. 27, 1916; Capt., Engrs., Casual, Aug. 2, 1916. Overseas service Feb. 29, 1918–June 2, 1919. Discharged June 2, 1919. Co. Comdr., 6th and 316th Engrs.; Graduate, School of Line, Langres, France; Div. Engr. Officer.

GLANDER, JOHN HENRY, JR.
Entered service June 7, 1918, as Ensign, C. E. C., U. S. N. R. F.; Lt., Jr. Grade, Dec. 5, 1918. Released from active service July 2, 1919. Constr. work at Washington, D. C., and Brooklyn, N. Y.

GLAZIER, WILLIAM LEONARD
Entered service July 31, 1917; Maj., Engrs., N. A., July 22, 1917. Overseas service July 10, 1918–July 29, 1919. Discharged Aug. 15, 1919. Organized and commanded 525th Engrs. Officier du Merite Agricole. One star.

GODDARD, LESLIE DREW
Entered service Dec. 28, 1917; Capt., E. O. R. C., June 30, 1917. Discharged Feb. 7, 1919. E. O. T. C., Camp Lee; Asst. to Supervising Const., Q. M., Raritan and Middletown Arsenals; Supervising Const. Q. M., in chg. constr. at permanent arsenals.

GODFREY, ALEXANDER HOLLIS
Entered service Jan. 29, 1918, as Chf. Yeoman, U. S. N. R. F.; Ensign, U. S. N. R. F., May 30, 1918. Released from active duty Aug. 1, 1919. Paymasters' School, Naval Academy; Supply and Disbursing Officer, Naval Plant, Ft. Worth, and Ammunition Plant, Portsmouth, Va.

GODFREY, STUART CHAPIN
Entered service June 15, 1905; Capt., C. of E., U. S. A., at declaration of war; Maj., C. of E., U. S. A., Aug. 10, 1917; Lt. Col., C. of E., U. S. A., Jan. 5, 1918; Col., Engrs., N. A., Aug. 1, 1918. Overseas service May 10, 1918–Aug. 1, 1919. Instr. at Ft. Leavenworth; with 318th Engrs.; Engr. Personnel Officer, A. E. F.; Asst. to Chf. Engr., 1st Army; Executive Officer for Chf. Engr., 3d Army; Div. Engr., 2d Div., and C. O., 2d Engrs., Meuse-Argonne offensive. Officier, Order of Palms, France. One star.

GOETHALS, GEORGE RODMAN
Entered service June 16, 1904; through all grades in C. of E., U. S. A., to Maj. Aug. 22, 1917; Lt. Col., Dec. 20, 1917; Col., Aug. 1, 1918. Overseas service Apr. 7, 1918–May 24, 1919. Engr. Officer, H. Q., 1st Army Artillery; in chg. gen. constr., Office, Chf. Engr., 1st Army; C. O., 316th Engrs.; Div. Engr., 91st Div.; Engr. Member, Bd. of Officers to report on system of seacoast defenses employed by England, France, Italy, Austria and Turkey. Citation from Gen. Pershing for meritorious service. Three stars.

GOETHALS, GEORGE WASHINGTON
Returned to active service from retired list Dec. 12, 1917; Maj. Gen., U. S. A. Returned to retired list at own request Mar. 4, 1919. Acting Q. M. Gen.; later, in addition, Director of Storage and Traffic; Asst. Chf. of Staff, Director of Purchase, Storage and Traffic. Distinguished Service Medal; Commandeur, Legion d'Honneur; Knight Commander, Order of St. Michael and St. George, Great Britain.

GOLDEN, WILLIAM ANTHONY
Entered service June 1, 1917, as Seaman, 1st Class, U. S. N. R. F.; Ensign, U. S. N. R. F., Sept., 1917. Released from active duty, Dec. 12, 1918. Executive Officer, U. S. S. *Mary Alice.* Silver Medal of Honor awarded under Act of Congress.

GOLDSMITH, CLARENCE
Entered service Apr. 6, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged Apr. 30, 1919. Advisory Engr. on fire prevention and Prin. Asst. to Advisory Engr. on water supply.

GOOCH, CARL JOSEPH
Entered service Feb. 18, 1918, as Pvt., Engrs., N. A.; Sgt., 1st Class, Engrs., N. A., Apr. 1, 1918; M. E., Sr. Grade, Engrs., N. A., June 1, 1918; 2d Lt., Engrs., U. S. A., May 17, 1919. Overseas service, June 15, 1918–June 13, 1919. Discharged July 2, 1919. With 32d Engrs. at Base Sec. No. 2, A. E. F., on dock site investigation and eng. work, St. Sulpice Yard.

GOODFELLOW, JAMES
Maj., Royal Engrs., B. E. F.* Killed Mar. 23, 1918, in action in France.

GOODMAN, BENJAMIN SETH
Entered service Dec. 11, 1917, as Pvt., Engrs., N. A.; Sgt., 1st Class, Engrs., N. A., May 29, 1918; 2d Lt., C. of E., U. S. A., Aug. 20, 1918; 1st Lt., C. of E., U. S. A., Oct. 1, 1918. Overseas service Jan. 15, 1918–Nov. 25, 1919. Discharged Nov. 28, 1919. With 447th Depot Detachment Engrs.; with 116th Engrs.; Asst. to Corps Engr., H. Q., 2d Army Corps; with 315th and 1st Engrs.; Instr., Ecole Polytechnique, Paris. Three stars.

GOODMAN, LEON
Entered service July 6, 1917, as Capt., E. O. R. C.; Maj., Engrs., U. S. A., Sept. 17, 1918. Overseas service Sept. 17, 1918–June 29, 1919. Discharged July 11, 1919. C. O., 109th Engrs.; various assignments on hosp., bldg., and road constr.

GORDON, JOHN BLAKE
U. S. A., A. E. F.*

GORDON, SAMUEL
Lt. Comdr., C. E. C., U. S. N.*

GORTON, WILLARD LIVERMORE
Entered service Sept. 19, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 11, 1918. Camp Humphreys.

GOSS, OLIVER PERRY MORTON
Entered service Dec. 22, 1917; Capt., Sig. C., N. A., Jan. 8, 1918. Discharged Jan. 13, 1919. In chg. Technical Sec., Spruce Production Div.

GOTWALS, JOHN CARL
Lt. Col., Engrs., U. S. A., A. E. F.*

GOUGH, WILLIAM JOSEPH
Entered service June 30, 1917, as Capt., E. O. R. C.; Maj., Engr. R. C., June, 1918. Discharged Mar. 12, 1919. With 316th, 318th, and 138th Engrs., and 3d Engr. Training Regt.

GOULD, WILLIAM TILLOTSON
Entered service May 22, 1917; Capt., E. O. R. C., June 28, 1917. Overseas service Dec. 14, 1917–Jan. 8, 1919. Discharged Jan. 13, 1919. With 301st Engrs., Camp Devens; in chg. erection of fuel oil and gasoline storage stations, Base Sec. No. 2, A. E. F.

GOW, CHARLES RICE
Entered service Feb. 8, 1918, as Maj. Q. M. C., N. A.; Lt. Col., Q. M. C., Constr. Div., U. S. A., May 3, 1919. Discharged Sept. 4, 1919. Const. Q. M., Army Supply Base, Temporary Warehouses, Boston, Mass.

GRADY, FRANK La SALLE
1st Lt., Inf., U. S. A., A. E. F.*

GRADY, PAUL LEO
1st Lt., Engrs., U. S. A.*

GRAETER, GEORGE CHRISTIAN
Entered service Apr. 16, 1917, as 1st Lt., E. O. R. C.; Capt., Engr., R. C., June 15, 1918. Overseas service Aug. 4, 1917–June 10, 1919. Discharged June 12, 1919. With Dept. Light Rys., Locating Engr., Resident Engr., Abainville Shops; Bn. Comdr. Two stars.

GRAFF, GEORGE WASHINGTON
Entered service June 24, 1918; Cpl., 155th Depot Brig., July 10, 1918. Discharged Nov. 29, 1918. Candidate, F. A. Training School.

GRANBERY, JULIAN HASTINGS
Entered service Sept. 15, 1916. Conducteur, S. S. U. 7, 2d French Army. Overseas service Sept. 15, 1916–Mar. 31, 1917. Discharged Mar. 31, 1917. With S. S. U., 21st Div.

GRANT, JOHN ROBERT
Entered service May 13, 1915, as 2d Lt., Royal Engrs., British Army; Lt., Jan. 1, 1916; Capt., Aug., 1916; Maj., Apr., 1918. Overseas service Mar. 2, 1915–Dec. 19, 1918. Discharged Apr. 29, 1919. Field Co. Officer; Adj., 24th Div.; C. O., 2d Field Co., 8th Div. Military Cross, Great Britain. Slightly wounded.

GRANT, KENNETH CROTHERS
Entered service Apr. 18, 1917, as Capt., E. O. R. C.; Maj., Q. M. C., Constr. Div., U. S. A., Oct. 17, 1918. Discharged Mar. 15, 1919. In chg. Engr. Sec. Constr. Div., Aviation Sec., Sig. C.; C. O., 499th Aero Squadron, Langley Field; in chg. constr. all camps and hosps. in northern U. S.

GRANT, ULYSSES S, 3d
Entered service June 13, 1899; through all grades in C. of E., U. S. A., to Maj. at declaration of war; Lt. Col., Aug. 5, 1917; Col., Dec. 20, 1917. Overseas service Jan. 12, 1918–Dec. 20, 1919. Office, Chf. of Engrs.; Office, Chf. of Staff; War College Div.; member American Sec., Supreme War Council; Asst. Commissioner of American-German Conference on Prisoners of War; member International Joint Secretariat, Peace Conference. Officier, Legion d'Honneur; Distinguished Service Medal; Officer, Order of St. Maurice and St. Lazarus, Italy; Commander, Order of St. Michael and St. George, Great Britain; Order of Solidaridad, Panama. Lys defensive. One star.

GRAVELL, WILLIAM HENRY
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 19, 1917; Maj., Engrs., U. S. A., Apr. 8, 1919. Overseas service Jan. 10, 1918–Nov. 15, 1919. Discharged Sept. 15, 1919. Gen. Staff, G. H. Q., and H. Q., S. O. S., A. E. F.

GRAVES, EDWARD MICHAEL
Entered service Apr., 1917, as Capt., E. O. R. C.; Maj., Engr. R. C., Dec., 1917. Overseas service June 30, 1918–Dec., 1918. Discharged Jan. 3, 1919. With 308th and 28th Engrs.; C. O., 529th Engrs.

GRAY, EARLE PIERCE
Entered service May 7, 1917; 1st Lt., E. O. R. C., Jan. 19, 1917. Overseas service Aug. 1, 1917–July 17, 1919. Discharged Aug. 9, 1919. With 16th, 21st, 302d and 310th Engrs.; Engr. School at Langres, France. Divisional Citation, 77th Div.; Order of St. Stanislaus, with sword and ribbon, Russia. Lys defensive; St. Mihiel and Meuse-Argonne offensives; Russian expedition. Four stars.

GRAY, WILLIAM BACON
Entered service Jan. 2, 1918; Maj., Q. M. C., Constr. Div., U. S. A., Sept. 24, 1918. Discharged July 17, 1919. H. Q., Constr. Div., Washington, D. C.; Const. Officer, Middletown Ord. Depot and Marlin Rockwell plant.

GREEN, CHARLES NEWTON
Entered service Dec. 28, 1917, as Maj., Engrs., N. A. Discharged Oct. 11, 1919. Camp Lee; Const. Q. M., Bag Loading Plant, Woodbury, N. J.; Washington, D. C.; Aberdeen Proving Grounds.

GREEN, CHARLES SAMUEL
Entered service Dec. 28, 1917; Capt., Engrs., N. A., Sept. 18, 1917. Overseas service Nov. 10, 1918–July 5, 1919. Discharged July 14, 1919. Instr., Camp Lee; Co. Comdr., 3d Engr. Training Regt., Camp Humphreys; C. O., 549th Enrgs., Service Bn.

GREEN, CLARENCE JASPER
Entered service July 25, 1917, as 2d Lt., C. A. C., N. G.; 1st Lt., C. A. C., N. G., Nov. 28, 1917; Capt., C. A. C., U. S. A., Sept. 13, 1918. Discharged Dec. 28, 1918. Artillery Engr., Coast Defenses, Columbia River; Regtl. Orientation Officer, 39th Artillery.

GREEN, FREDERICK WILLIAM
Entered service May 7, 1917; Capt., E. O. R. C., Feb. 16, 1917; Maj., R. T. C., N. A. May 18, 1918; Lt. Col., R. T. C., U. S. A., Oct. 1, 1918. Overseas service July 28, 1917–Mar. 7, 1919. Discharged Mar. 13, 1919. Co. Comdr., 12th Engrs.; Supt. Transport Service at Brest and St. Nazaire, France; C. O., 1st Grand Div. T. C. Distinguished Service Medal; Officier, Legion d'Honneur.

GREENE, FREDERICK STUART
Entered service May 19, 1917; Capt., E. O. R. C., Apr. 21, 1917. Overseas service Mar. 29, 1918–Jan. 12, 1919. Discharged Mar. 11, 1919. Bn. Comdr., 302d Engrs. Four stars.

GREENFIELD, ROBERT ARTHUR
Entered service May 15, 1917. Capt., E. O. R. C., June, 1917; Maj., Engrs., N. A., July, 1918. With 303d Engrs.

GREGORY, JOHN HERBERT
Entered service Nov. 8, 1918; Capt., San. C., U. S. A., Nov. 2, 1918. Discharged Dec. 20, 1918. Camp Greenleaf.

GREGORY, LUTHER ELWOOD
Entered service Apr. 5, 1898; through all grades in C. E. C., U. S. N., to Capt., July 1, 1917. Public Works Officer, Puget Sound Navy Yard and Boston Navy Yard.

GREGORY, WILLIAM BENJAMIN
Entered service Sept. 26, 1917; Maj., E. O. R. C., March 10, 1917. Overseas service Oct. 3, 1917–Jan. 30, 1919. Discharged Feb. 6, 1919. Water Supply Office, H. Q., Div. Constr. and Forestry, A. E. F. Citation from Gen. Pershing for meritorious service.

GREHAN, BERNARD HENRY
Entered service May 12, 1917; 2d Lt., E. O. R. C., June 28, 1917; 1st Lt., Engr. R. C., Dec. 31, 1917. Overseas service Aug. 25, 1918–July 6, 1919. Discharged July 25, 1919. With 312th Engrs.

GRIEST, MAURICE
Entered service Sept. 22, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 27, 1918. With R. T. C., Washington, D. C.; 4th Engr. Training Regt., Camp Humphreys.

GRIFFIN, JOHN ALDEN
Entered service May 8, 1917; Capt., Engrs., N. A., Aug. 23, 1917; Maj., Engrs., U. S. A., Oct., 1918. Overseas service July 6, 1918–Sept. 11, 1918. Discharged Jan. 25, 1919. With 316th and 607th Engrs.

GRIGSBY, WALTER BERTON
Entered service Sept. 4, 1917; 2d Lt., E. O. R. C., July 2, 1917; 1st Lt., Engrs., N. A., Dec. 11, 1917. Discharged Dec. 18, 1919. With 305th Engrs.; with 9th Engrs. as Adj., Supply Officer and Topographic Officer.

GRIMES, JAMES EDWARD
Entered service Mar. 23, 1918, as Capt., Q. M. C., Constr. Div., N. A. Discharged Mar. 28, 1919. Asst. to Const. Q. M., Aberdeen Proving Grounds and T. A. Gillespie Loading Co., Morgan, N. J.; Asst. to Officer in Chg. of Utilities, Camp Grant.

GRINDROD, ERVIN SUTTON
Entered service Sept. 28, 1917; 1st Lt., Engrs., N. A., Aug. 15, 1917; Capt., Engrs., U. S. A., May 7, 1919. Overseas service May 10, 1918–July 5, 1919. Discharged July 29, 1919. With 103d Engrs.; Bn. Adj. and Supply Officer, 33d Engrs.; Engr. Officer in chg. constr., Div. of Constr. and Forestry, A. E. F. Citation from Gen. Pershing for meritorious service.

GRISWOLD, HARRY TODD
Entered service July 28, 1917, as 2d Lt., C. A. C., N. A.; 1st Lt., C. A. C., N. A., Nov., 1917. Overseas service Mar. 28, 1918–Jan. 28, 1919. Discharged Jan. 28, 1919. With H. Q. Co., 56th Regt., C. A. C.; Artillery Intelligence Officer, G-2, 1st Army, A. E. F.

GRISWOLD, HECTOR CLINTON
Entered service June 15, 1917, as Lt., Jr. Grade, C. E. C., U. S. N.; Lt., C. E. C., U. S. N., Oct. 15, 1917. In chg. constr. at Washington, D. C.; member Examining Bd.; in chg. constr. at naval ammunition depots in New York Dist., and Public Works Office, Fleet Supply Base, South Brooklyn, N. Y.

GRODSKE, WALTER JOHN
Capt., Engrs., U. S. A.*

GROSS, FREDERICK HENRY
Entered service May 10, 1917, as 2d Lt., E. O. R. C.; 1st Lt., Engrs., U. S. A., Sept., 1918. Overseas service May 24, 1918–June 23, 1919. Discharged July 3, 1919. With 303d and 113th Engrs.

GROSS, HENRY McCORMICK
Entered service June, 1917, as 1st Lt., Inf.; 1st Lt., Machine Gun Bn., Aug. 25, 1917; Capt., Inf., U. S. A., Oct., 1918. Overseas service Oct. 1917–Apr. 1919. Discharged May, 1919. With 8th Penna. Inf.; 149th Machine Gun Bn.; Chf. Machine Gun Instr., 92d Div.; Aide to Maj.-Gen. William H. Hay. Cited in Brig. Operations Report, 55th Inf. Brig. One star.

GRUNAUER, MORTIMER
Entered service Apr. 4, 1918, as Pvt., F. A., N. A.; Sgt., F. A., N. A., Aug. 1, 1918. Overseas service Apr. 24, 1918–July 18, 1919. Discharged July 23, 1919. With 304th F. A.; Instr. in Orientation, F. A. School, Camp de Souge, France. One star.

GUILLEMETTE, JOSEPH DYDIME
Entered service Apr. 11, 1918, as Pvt., 1st Class, Aviation Section, Sig. C., N. A.; 2d Lt., A. S., N. A., July 19, 1918. Overseas service July 30, 1918–Jan. 7, 1919. Discharged Jan. 7, 1919. Office, Chf. Sig. Officer, H. Q., S. O. S., A. E. F.

GUNTHER, HERMAN DIETRICH
Entered service Apr. 2, 1918, as Pvt., Inf., N. A. Overseas service June 23, 1918–Mar. 23, 1919. Discharged Apr. 7, 1919. With 147th Inf. Two stars.

GUPPY, BENJAMIN WILDER
Entered service May 16, 1917, as Maj., E. O. R. C.; Lt. Col., Engrs., N. A., Mar. 19, 1918. Overseas service July 27, 1917–Feb. 9, 1919. Discharged Feb. 11, 1919. C. O., 14th Engrs.; 2d in command, Camp Hunt; Supt., Port Terminal Facilities, St. Nazaire, France; British front near Calais; Marne Salient; with 1st Army, 3d Corps, A. E. F. Three stars.

GUPTILL, JOSEPH RICKER
Entered service Nov. 3, 1917; Pvt., Engrs., N. A., Nov. 14, 1917. Overseas service, Mar. 31, 1918–June 16, 1919. Discharged June 27, 1919. With 23d Engrs., gen. constr. highways and railroads; Meuse-Argonne offensive. One star.

GUTMAN, DAVID
Entered service Jan. 4, 1918; Maj., Engrs., N. A., Sept. 17, 1917. Discharged Dec. 28, 1918. E. O. T. C., Camp Lee; Constr. Div., Washington, D. C.; Bn. Comdr., 4th Engr. Training Regt., and in chg. of constr., Camp Humphreys; with 5th Prov. Bn., in chg. of constr., Camp Benjamin Harrison.

HAAS, PHILIP LIPPMAN
Capt., Engrs., U. S. A.*

HADLEY, ALLEN STACY
Entered service Sept. 5, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 10, 1919. With 139th and 218th Engrs.; detailed to M. T. C.

HAGUE, WILLIAM
Entered service 1917 as 1st Lt., Engr. R. C. With 116th Engrs., at the Presidio, Vancouver Barracks, Camps American Lake, Charlotte, Mineola, and in France. Died Jan. 2, 1918, in active service in France.

HAINS, PETER CONOVER
Entered service June 1, 1857; through various grades to Maj. Gen., U. S. A. (*Retired*). Assigned to active duty Sept. 19, 1917. In chg., River and Harbor Works and Defenses, Norfolk, Va., and Chesapeake Bay.

HALCOMBE, NORMAN MARSHALL
Entered service May 1, 1915, as 2d Lt., Royal Engrs., British Army; Lt., Royal Engrs., June, 1915; Capt., Royal Engrs., Aug. 4, 1915; Capt., Royal Flying C., Nov. 1, 1916; Maj., Royal Flying C., Nov. 16, 1917. Overseas service, 1915–1919. Co. Comdr., 31st Cheshire Field Co., R. E., 21st Welsh Field Co., R. E., and 21st Cheshire Field Co., R. E. (Gallipoli); special evacuation work in Egypt; special transport work in Western Desert, Egypt and Sinai; organized and developed Engine Repair Depots, Royal Flying Corps, in Egypt and Palestine; with Technical Dept., Parachute Sec., Aircraft Production, London, England; Staff Officer, Indian Air Force, India. Mentioned three times in dispatches. Died Feb. 15, 1919, in active service at Port Said, Egypt.

HALDEMAN, WALTER STANLEY
Entered service June 11, 1917, as Capt., E. O. R. C. Discharged Dec. 15, 1918. Acting Bn. Adj., 314th Engrs.; Instr., E. O. T. S., Camp Funston; Co. Comdr., 3d Engrs. Training Regt.; Bn. Adj., 606th Engrs.

HALE, RICHARD KING
Entered service July 25, 1917, as Lt. Col., F. A., N. A.; Col., F. A., N. A., Nov. 7, 1917. Overseas service Sept. 9, 1917–Apr. 4, 1919. Discharged June 14, 1919. C. O., 103d F. A.; with 101st F. A.; G. S. College overseas; Asst. Chf. of Staff (G-Q), 2d Army Corps; Gen. Staff 2d Army Corps with British Army; Asst. Chf. of Staff, American Embarkation Center; Chf. of Staff, 26th Div.

HALL, BENJAMIN MORTIMER, JR.
Entered service May 14, 1917; 1st Lt. Engrs., N. A., Aug. 1, 1917; Capt., Engrs., U. S. A., Nov. 6, 1918. Overseas service Oct. 31, 1917–June 17, 1919. Discharged June 28, 1919. Asst. to Officer in Chg. Water Supply, Base Sec. No. 1, A. E. F.; Co. Comdr., 26th Engrs., during St. Mihiel offensive; Staff of Chf. Engr., 2d Army, as Asst. Water Supply Officer; Acting Officer in Chg. Water Supply Sec., Office of Chf. Engr., A. E. F. One star.

HALL, CHARLES LACEY
Through all grades in C. of E., U. S. A., to Maj., May 15, 1917; Lt. Col., Dec. 20, 1917; Col. (temporary), Aug. 1, 1918. Overseas service July 28, 1917–June 28, 1919. Reverted to permanent grade Oct. 6, 1919. Adj., 12th Engrs.; G. S., A. E. F.; Asst. Sec. Engr., Advance Sec., A. E. F. Cited by Comdr. in Chf. for services as Chf., G-2-c, 1st Army, and in connection with organization of sound and flash ranging troops. Four stars.

HALL, GILBERT G.
Entered service Sept. 2, 1917; 1st Lt., Engrs., N. A., Dec. 6, 1917. Overseas service, Jan. 2, 1918–July 29, 1919. Discharged Aug. 16, 1919. With 20th Engrs. as Ry. Transfer Officer, and Camp Comdr., "Burned Area" Dist., Pontenx-les-Forges, France.

HALL, JULIUS REED
Entered service Sept. 25, 1917, as 1st Lt., E. O. R. C.; Capt., Engrs., U. S. A., May 12, 1919. Overseas service May 5, 1918–July 12, 1919. Discharged Aug. 6, 1919. With 515th Engrs. on Depot Constr., Gievres, France.

HALL, LOUIS WELLS
Entered service July 19, 1917, as Capt. E. O. R. C. Overseas service July 23, 1917–Feb. 11, 1919. Discharged Feb. 17, 1919. Assigned to engrs. in 1st Div., preparing camps, hosps., etc.; Office, Chf. Engr., A. E. F., in chg. Drafting and Map Production Dept., Div Constr. and Forestry.

HALL, OLIVER ANTRUM
Entered service Nov. 26, 1917; Capt., Engrs., N. A., Dec. 6, 1917. Overseas service Mar. 28, 1918–July 27, 1919. With 23d Engrs.; Engr. Officer in chg. design and constr. ord. repair shops and bldgs. at Clermont-Ferrand, France; with regt. at front as Liaison Officer between 1st, 3d and 5th Corps, and H. Q., 1st Army Roads Office; assigned to C. W. S.; Instr. C. W. S. school at Lakehurst Proving Grounds, N. J. Two stars.

HALL, WARREN ESTERLY
Entered service Sept. 25, 1917; Capt., Engrs., N. A., Nov. 27, 1917. Overseas service Jan. 4, 1918–Mar. 25, 1919. Discharged July 18, 1919. With 506th Engrs.; on heavy railroad constr., Base Sec. No. 2, A. E. F.; Asst. Water Supply Engr. and Water Supply Engr., Base Sec. No. 2, A. E. F. One star.

HALLIHAN, JOHN PHILIP
Entered service Aug. 3, 1918; Maj., Engrs., U. S. A., Aug. 17, 1918. Overseas service Oct. 17, 1918–Mar. 6, 1919. Discharged Mar. 12, 1919. C. O., 70th Engrs. and Post Comdr., Ft. Douglas; in Engr. Supply Sec., Div. Military Eng. and Engr. Supplies, A. E. F.; in Transportation Sec. Engr. Dept., American Commission to Negotiate Peace.

HALSEMA, EUSEBIUS JULIUS
Capt., Engrs., U. S. A.*

HAMILTON, EDWARD PARMELEE
Entered service Aug. 15, 1917, as 2d Lt., F. A., N. A.; 1st Lt., F. A., N. A., Jan. 2, 1918. Overseas service Apr. 24, 1918–Apr. 29, 1919. Discharged May 10, 1919. With 306th F. A. as Battery Officer, Regimental Reconnaissance and Personnel Adj.; Baccarat and Vesle Sectors, Oise-Aisne and Meuse-Argonne offensives. Two stars. Prisoner of war, Sept. 28 to Nov. 29, 1918.

HAMMILL, HAROLD BERNARD
Entered service Apr. 15, 1918, as Lt., Jr. Grade, C. E. C., U. S. N. R. F. Overseas service May 16, 1918–Feb. 1, 1919. Released from active duty Mar. 17, 1919. With unit on constr. of high-power radio plant at Croix d'Huis, France.

HAMMOND, JOHN MILLER
Entered service May 16, 1918, as Capt., Q. M. C., Constr. Div., N. A. On constr. projects at New Orleans Army Supply Base, Rock Island Arsenal and Honolulu, Hawaii; Const. Q. M. at Schofield Barracks, Hawaii.

HANAVAN, WILLIAM LAWRENCE
Entered service May 8, 1917; 1st Lt., Inf., N. G., Nov. 6, 1916; Capt., Inf., N. A., July 12, 1918. Overseas service Sept. 8, 1917–Jan. 5, 1919. Discharged May 5, 1919. Inf. Officers' School, France; Co. Comdr. and Bn. Adj., 9th Inf.; Machine Gun School, Gondrecourt, France; Personnel Adj. and Comdr. railhead detachment, 9th Inf.

HANCOCK, HENRY SYDNEY, JR.
Entered service Dec. 9, 1915, British Army; 2d Lt. Royal Engrs., June 25, 1916; Lt., Royal Engrs., Apr. 18, 1917. Overseas service Oct. 28, 1915–Oct. 6, 1919. Discharged Oct. 6, 1919. With 254th Tunneling Co.; Field Engr., 1st Corps, in chg. support lines and corps defenses, heavy gun emplacements, water supplies, encampments, etc. (Acting Maj.); with 170th Tunneling Co. Wounds—"None sufficiently serious to prevent my carrying on."

HANIQUE, JULES EDMOND
Entered service May 10, 1917; Capt., E. O. R. C., June 20, 1917; Maj., Engr. R. C., May 22, 1918. Overseas service July 13, 1918–Aug. 14, 1919. Discharged Aug. 22, 1919. Adj., 316th Engrs., Camp Lewis; with 537th Engrs.; Acting Corps Road Engr. during Meuse-Argonne offensive; Liaison Officer, French Staff, Verdun; Dist. Road Engr. Citation for efficiency during Argonne drive. Two stars.

HANNAN, DAVID EDWARD
Capt., Engrs., U. S. A., A. E. F.*

HANSEN, PAUL
Entered service May 7, 1917; Capt., E. O. R. C., May 1, 1917. Overseas service Aug. 5, 1917–Apr. 7, 1919. Discharged Apr. 15, 1919. On staff duty at G. H. Q., A. E. F.; H. Q., Advance Sec., A. E. F.; with 1st and 2d Armies and with Peace Commission.

GARNETT, BENJAMIN JAY
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., June, 1917. Overseas service Jan. 26, 1918–Mar. 1, 1919. Discharged Mar. 5, 1919. Constr., Advance Sec., S. O. S., A. E. F.

GARRETT, ROBERT PEEL
Entered service July 16, 1918, as Capt., Ord. C., N. A. Discharged July 9, 1919. On duty with Ry. and Seacoast Carriage Sec., Arty. Div., Ord. Dept., Washington, D. C.

GATENS, RAY STUART
Entered service Mar. 1, 1918, as 1st Lt., Engrs., N. A.; Capt., Engrs., U. S. A., Jan. 26, 1919. Overseas service May 10, 1918–May 24, 1919. Discharged June 23, 1919. With 42d, 43d and 103d Engrs.; 1st Army, A. E. F.

GAUMER, ALBERT WESLEY
Entered service Aug. 12, 1918; 1st Lt., Engrs., U. S. A., Nov. 14, 1918. Discharged Jan. 17 1919. With 4th and 8th Engr. Training Regts; Co. Comdr., 4th Engr. Training Regt.

GAUSMANN, ROY WARNER
Entered service May 8, 1917; Capt., E. O. R. C., Feb. 23, 1917; Maj., Engr. R. C., Jan. 7, 1918. Overseas service May 8, 1918–Jan. 8, 1919. Discharged Jan. 11, 1919. Co. Comdr., and Bn. Comdr., 303d Engrs.; British front, Strazelle Sector; St. Mihiel and Meuse-Argonne offensives. Three stars. Wounded July 15, 1918.

GAYLER, ERNEST ROTTECK
Entered service Oct., 1902; through all grades in C. E. C., U. S. N., to present grade of Comdr.

GEIB, GEORGE ALBERT
Entered service May 11, 1917; 1st Lt., E. O. R. C., June 22, 1917; Capt., E. O. R. C., Aug. 15, 1917. Overseas service June 22, 1918–July 1, 1919. Discharged July 24, 1919. Adj., and Topographic Officer, 313th Engrs.; Co. Comdr., 527th Engrs.; St. Mihiel and Meuse-Argonne offensives. Two stars.

GENUNG, JAMES HOLCOMBE, JR.
Entered service May 26, 1917, as 2d Lt., E. O. R. C.; 2d Lt., F. A., N. A., Aug. 8, 1917; 1st Lt., F. A., N. A., Nov. 15, 1917; Capt., F. A., N. A., July, 1918; Maj., F. A., U. S. A., Sept., 1918. Overseas service May 1, 1918–Sept. 10, 1918. Co. Comdr., and Bn. Comdr., 21st F. A.; Bn. Comdr. 69th F. A.; C. O. 2d and 71st F. A.; Military Observer, 21st French Army Corps, Verdun and Champagne. Two stars.

GEORGE, HOWARD HOWELL
Entered service May 8, 1917; 1st Lt., E. O. R. C., Jan. 16, 1917; Capt., E. O. R. C., Aug. 15, 1917. Overseas service June 30, 1918–Jan. 26, 1919. Discharged Jan. 29, 1919. Co. Comdr., 305th and 55th Engrs.; Engr. Officer in chg. bldg. constr., Chateaureux Storage Depot Project; with Sec. Engr., Base Sec. No. 4, A. E. F.

GERNER, ANSON JOHN
Pvt., E. O. T. S., U. S. A.*

GETTY, LORENZO TODD
Entered service Nov. 2, 1917; 1st Lt., Aviation Sec., Sig. C., N. A., Oct. 25, 1917. Capt., A. S., U. S. A., Oct. 15, 1918; Maj., A. S., Feb. 21, 1919. Overseas service, Dec. 3, 1917–Apr. 20, 1919. Discharged May 7, 1919. Adj., 75th Aero Squadron; Constr. Supply Officer, Asst. Adj., Adj., and Executive Officer, A. S. Production Center No. 2. Chevalier, Ordre de l'Etoile Noire.

GIBBS, ELBERT ALLAN
Entered service Apr. 25, 1917; Capt., E. O. R. C., May 19, 1917; Maj., E. O. R. C., July 29, 1917; Lt. Col., Engrs., N. A., June 17, 1918; Col., Engrs., U. S. A., Feb. 13, 1919. Overseas service July 9, 1917–July 7, 1919. Discharged July 24, 1919. With 5th, 15th, 32d, and 311th Engrs.; H. Q., S. O. S., A. E. F.; Chf., Gen. Constr. Sec., Div. of Constr. and Forestry, A. E. F.; served on various boards. Citation from Gen. Pershing for meritorious service; Legion d'Honneur; Distinguished Service Medal. One star.

GIBBS, WILLIAM WETMORE†
1st Lt., Engrs., U. S. A., A. E. F.*

GIBSON, THOMAS FENNER
Entered service Sept. 3, 1918, as Pvt., Inf., U. S. A.; Cpl., Inf., U. S. A., Nov. 1, 1918. Discharged Jan. 30, 1919. With 63d Inf.

GIESEN, WALTER EDWARD
Entered service July 23, 1918, as Pvt., U. S. M. C. Discharged Jan. 23, 1919. With 15th Marines.

GIESTING, FRANK ALEXANDER
Entered service May, 1917, as Maj., E. O. R. C.; Lt. Col., Engrs., N. A., Aug., 1918; Col., Engrs., U. S. A., Nov. 15, 1918. Overseas service Mar. 1917–May, 1919. Discharged May, 1919. With 302d Engrs. Croix de Guerre.

GIFFELS, WILLIAM CHARLES
Entered service May 14, 1917; 1st Lt., Engrs., N. A., Aug. 26, 1917. Overseas service July 16, 1918–July 18, 1919. Discharged Aug. 6, 1919. With 310th Engrs. in North Russia Military Cross, Great Britain; Order of St. Anne, Russia.

GILES, JOHN ANGUS
Entered service Sept. 9, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Jan. 31, 1919. Asst. Officer in Chg. Utilities, Camp Upton; Officer in Chg. Utilities, Gen Hosp. No. 2; Asst. to Officer in Chg., Port Utilities, New York City.

† Died Jan. 13, 1919.

GILMAN, EDGAR DOW
Entered service Mar. 15, 1918, as 1st Lt., C. W. S., N. A. Discharged Apr. 30, 1919. With 473d Engrs.; Chf. Gas Officer, Camp Greene; member Bd. of Officers for Determining Land Damage Claims; Camp Exchange Officer; Camp Adj.

GLADDING, JAMES NICKERSON
Entered service Sept. 27, 1916; Capt., Engrs., Casual, Aug. 2, 1916. Overseas service Feb. 29, 1918–June 2, 1919. Discharged June 2, 1919. Co. Comdr., 6th and 316th Engrs.; Graduate, School of Line, Langres, France; Div. Engr. Officer.

GLANDER, JOHN HENRY, JR.
Entered service June 7, 1918, as Ensign, C. E. C., U. S. N. R. F.; Lt., Jr. Grade, Dec. 5, 1918. Released from active service July 2, 1919. Constr. work at Washington, D. C., and Brooklyn, N. Y.

GLAZIER, WILLIAM LEONARD
Entered service July 31, 1917; Maj., Engrs., N. A., July 22, 1917. Overseas service July 10, 1918–July 29, 1919. Discharged Aug. 15, 1919. Organized and commanded 525th Engrs. Officier du Merite Agricole. One star.

GODDARD, LESLIE DREW
Entered service Dec. 28, 1917; Capt., E. O. R. C., June 30, 1917. Discharged Feb. 7, 1919. E. O. T. C., Camp Lee; Asst. to Supervising Const., Q. M., Raritan and Middletown Arsenals; Supervising Const. Q. M., in chg. constr. at permanent arsenals.

GODFREY, ALEXANDER HOLLIS
Entered service Jan. 29, 1918, as Chf. Yeoman, U. S. N. R. F.; Ensign, U. S. N. R. F., May 30, 1918. Released from active duty Aug. 1, 1919. Paymasters' School, Naval Academy; Supply and Disbursing Officer, Naval Plant, Ft. Worth, and Ammunition Plant, Portsmouth, Va.

GODFREY, STUART CHAPIN
Entered service June 15, 1905; Capt., C. of E., U. S. A., at declaration of war; Maj., C. of E., U. S. A., Aug. 10, 1917; Lt. Col., C. of E., U. S. A., Jan. 5, 1918; Col., Engrs., N. A., Aug. 1, 1918. Overseas service May 10, 1918–Aug. 1, 1919. Instr. at Ft. Leavenworth; with 318th Engrs.; Engr. Personnel Officer, A. E. F.; Asst. to Chf. Engr., 1st Army; Executive Officer for Chf. Engr., 3d Army; Div. Engr., 2d Div., and C. O., 2d Engrs., Meuse-Argonne offensive. Officier, Order of Palms, France. One star.

GOETHALS, GEORGE RODMAN
Entered service June 16, 1904; through all grades in C. of E., U. S. A., to Maj. Aug. 22, 1917; Lt. Col., Dec. 20, 1917; Col., Aug. 1, 1918. Overseas service Apr. 7, 1918–May 24, 1919. Engr. Officer, H. Q., 1st Army Artillery; in chg. gen. constr., Office, Chf. Engr., 1st Army; C. O., 316th Engrs.; Div. Engr., 91st Div.; Engr. Member, Bd. of Officers to report on system of seacoast defenses employed by England, France, Italy, Austria and Turkey. Citation from Gen. Pershing for meritorious service. Three stars.

GOETHALS, GEORGE WASHINGTON
Returned to active service from retired list Dec. 12, 1917; Maj. Gen., U. S. A. Returned to retired list at own request Mar. 4, 1919. Acting Q. M. Gen.; later, in addition, Director of Storage and Traffic; Asst. Chf. of Staff, Director of Purchase, Storage and Traffic. Distinguished Service Medal; Commandeur, Legion d'Honneur; Knight Commander, Order of St. Michael and St. George, Great Britain.

GOLDEN, WILLIAM ANTHONY
Entered service June 1, 1917, as Seaman, 1st Class, U. S. N. R. F.; Ensign, U. S. N. R. F., Sept., 1917. Released from active duty, Dec. 12, 1918. Executive Officer, U. S. S. *Mary Alice*. Silver Medal of Honor awarded under Act of Congress.

GOLDSMITH, CLARENCE
Entered service Apr. 6, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged Apr. 30, 1919. Advisory Engr. on fire prevention and Prin. Asst. to Advisory Engr. on water supply.

GOOCH, CARL JOSEPH
Entered service Feb. 18, 1918, as Pvt., Engrs., N. A.; Sgt., 1st Class, Engrs., N. A., Apr. 1, 1918; M. E., Sr. Grade, Engrs., N. A., June 1, 1918; 2d Lt., Engrs., U. S. A., May 17, 1919. Overseas service, June 15, 1918–June 13, 1919. Discharged July 2, 1919. With 32d Engrs. at Base Sec. No. 2, A. E. F., on dock site investigation and eng. work, St. Sulpice Yard.

GOODFELLOW, JAMES
Maj., Royal Engrs., B. E. F.* Killed Mar. 23, 1918, in action in France.

GOODMAN, BENJAMIN SETH
Entered service Dec. 11, 1917, as Pvt., Engrs., N. A.; Sgt., 1st Class, Engrs., N. A., May 29, 1918; 2d Lt., C. of E., U. S. A., Aug. 20, 1918; 1st Lt., C. of E., U. S. A., Oct. 1, 1918. Overseas service Jan. 15, 1918–Nov. 25, 1919. Discharged Nov. 28, 1919. With 447th Depot Detachment Engrs.; with 116th Engrs.; Asst. to Corps Engr., H. Q., 2d Army Corps; with 315th and 1st Engrs.; Instr., Ecole Polytechnique, Paris. Three stars.

GOODMAN, LEON
Entered service July 6, 1917, as Capt., E. O. R. C.; Maj., Engrs., U. S. A., Sept. 17, 1918. Overseas service Sept. 17, 1918–June 29, 1919. Discharged July 11, 1919. C. O., 109th Engrs.; various assignments on hosp., bldg., and road constr.

GORDON, JOHN BLAKE
U. S. A., A. E. F.*

GORDON, SAMUEL
Lt. Comdr., C. E. C., U. S. N.*

GORTON, WILLARD LIVERMORE
Entered service Sept. 19, 1918 as Capt., Engrs., U. S. A. Discharged Dec. 11, 1918. Camp Humphreys.

GOSS, OLIVER PERRY MORTON
Entered service Dec. 22, 1917; Capt., Sig. C., N. A., Jan. 8, 1918. Discharged Jan. 13, 1919. In chg. Technical Sec., Spruce Production Div.

GOTWALS, JOHN CARL
Lt. Col., Engrs., U. S. A., A. E. F.*

GOUGH, WILLIAM JOSEPH
Entered service June 30, 1917, as Capt., E. O. R. C.; Maj., Engr. R. C., June, 1918. Discharged Mar. 12, 1919. With 316th, 318th, and 138th Engrs., and 3d Engr. Training Regt.

GOULD, WILLIAM TILLOTSON
Entered service May 22, 1917; Capt., E. O. R. C., June 28, 1917. Overseas service Dec. 14, 1917–Jan. 8, 1919. Discharged Jan. 13, 1919. With 301st Engrs., Camp Devens; in chg. erection of fuel oil and gasoline storage stations, Base Sec. No. 2, A. E. F.

GOW, CHARLES RICE
Entered service Feb. 8, 1918, as Maj. Q. M. C., N. A.; Lt. Col., Q. M. C., Constr. Div., U. S. A., May 3, 1919. Discharged Sept. 4, 1919. Const. Q. M., Army Supply Base Temporary Warehouses, Boston, Mass.

GRADY, FRANK La SALLE
1st Lt., Inf., U. S. A., A. E. F.*

GRADY, PAUL LEO
1st Lt., Engrs., U. S. A.*

GRAETER, GEORGE CHRISTIAN
Entered service Apr. 16, 1917, as 1st Lt., E. O. R. C.; Capt., Engr., R. C., June 15, 1918. Overseas service Aug. 4, 1917–June 10, 1919. Discharged June 12, 1919. With Dept. Light Rys., Locating Engr., Resident Engr., Abainville Shops; Bn. Comdr. Two stars.

GRAFF, GEORGE WASHINGTON
Entered service June 24, 1918; Cpl., 155th Depot Brig., July 10, 1918. Discharged Nov. 29, 1918. Candidate, F. A. Training School.

GRANBERY, JULIAN HASTINGS
Entered service Sept. 15, 1916. Conducteur, S. S. U. 7, 2d French Army. Overseas service Sept. 15, 1916–Mar. 31, 1917. Discharged Mar. 31, 1917. With S. S. U., 21st Div.

GRANT, JOHN ROBERT
Entered service May 13, 1915, as 2d Lt., Royal Engrs., British Army; Lt., Jan. 1, 1916; Capt., Aug., 1916; Maj., Apr., 1918. Overseas service Mar. 2, 1915–Dec. 19, 1918. Discharged Apr. 29, 1919. Field Co. Officer; Adj., 24th Div.; C. O., 2d Field Co., 8th Div. Military Cross, Great Britain. Slightly wounded.

GRANT, KENNETH CROTHERS
Entered service Apr. 18, 1917, as Capt., E. O. R. C.; Maj., Q. M. C., Constr. Div., U. S. A., Oct. 17, 1918. Discharged Mar. 15, 1919. In chg. Engr. Sec. Constr. Div., Aviation Sec., Sig. C.; C. O., 499th Aero Squadron, Langley Field; in chg. constr. all camps and hosps. in northern U. S.

GRANT, ULYSSES S, 3d
Entered service June 13, 1899; through all grades in C. of E., U. S. A., to Maj. at declaration of war; Lt. Col., Aug. 5, 1917; Col., Dec. 20, 1917. Overseas service Jan. 12, 1918–Dec. 20, 1919. Office, Chf. of Engrs.; Office, Chf. of Staff; War College Div.; member American Sec., Supreme War Council; Asst. Commissioner of American-German Conference on Prisoners of War; member International Joint Secretariat, Peace Conference. Officier, Legion d'Honneur; Distinguished Service Medal; Officer, Order of St. Maurice and St. Lazarus, Italy; Commander, Order of St. Michael and St. George, Great Britain; Order of Solidaridad, Panama. Lys defensive. One star.

GRAVELL, WILLIAM HENRY
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 19, 1917; Maj., Engrs., U. S. A., Apr. 8, 1919. Overseas service Jan. 10, 1918–Nov. 15, 1919. Discharged Sept. 15, 1919. Gen. Staff, G. H. Q., and H. Q., S. O. S., A. E. F.

GRAVES, EDWARD MICHAEL
Entered service Apr., 1917, as Capt., E. O. R. C.; Maj., Engr. R. C., Dec., 1917. Overseas service June 30, 1918 Dec., 1918. Discharged Jan. 3, 1919. With 308th and 28th Engrs.; C. O., 529th Engrs.

GRAY, EARLE PIERCE
Entered service May 7, 1917; 1st Lt., E. O. R. C., Jan. 19, 1917. Overseas service Aug. 1, 1917–July 17, 1919. Discharged Aug. 9, 1919. With 16th, 21st, 302d and 310th Engrs.; Engr. School at Langres, France. Divisional Citation. 77th Div.; Order of St. Stanislaus, with sword and ribbon, Russia. Lys defensive; St. Mihiel and Meuse-Argonne offensives; Russian expedition. Four stars.

GRAY, WILLIAM BACON
Entered service Jan. 2, 1918; Maj., Q. M. C., Constr. Div., U. S. A., Sept. 24, 1918. Discharged July 17, 1919. H. Q., Constr. Div., Washington, D. C.; Const. Officer, Middletown Ord. Depot and Marlin Rockwell plant.

GREEN, CHARLES NEWTON
Entered service Dec. 28, 1917, as Maj., Engrs., N. A. Discharged Oct. 11, 1919. Camp Lee; Const. Q. M., Bag Loading Plant, Woodbury, N. J.; Washington, D. C.; Aberdeen Proving Grounds.

GREEN, CHARLES SAMUEL
Entered service Dec. 28, 1917; Capt., Engrs., N. A., Sept. 18, 1917. Overseas service Nov. 10, 1918–July 5, 1919. Discharged July 14, 1919. Instr., Camp Lee; Co. Comdr. 3d Engr. Training Regt., Camp Humphreys; C. O., 549th Enrgs., Service Bn.

GREEN, CLARENCE JASPER
Entered service July 25, 1917, as 2d Lt., C. A. C., N. G.; 1st Lt., C. A. C., N. G., Nov. 28, 1917; Capt., C. A. C., U. S. A., Sept. 13, 1918. Discharged Dec. 28, 1918. Artillery Engr., Coast Defenses, Columbia River; Regtl. Orientation Officer, 39th Artillery.

GREEN, FREDERICK WILLIAM
Entered service May 7, 1917; Capt., E. O. R. C., Feb. 16, 1917; Maj., R. T. C., N. A. May 18, 1918; Lt. Col., R. T. C., U. S. A., Oct. 1, 1918. Overseas service July 28, 1917–Mar. 7, 1919. Discharged Mar. 13, 1919. Co. Comdr., 12th Engrs.; Supt. Transport Service at Brest and St. Nazaire, France; C. O., 1st Grand Div. T. C. Distinguished Service Medal; Officier, Legion d'Honneur.

GREENE, FREDERICK STUART
Entered service May 19, 1917; Capt., E. O. R. C., Apr. 21, 1917. Overseas service Mar. 29, 1918–Jan. 12, 1919. Discharged Mar. 11, 1919. Bn. Comdr., 302d Engrs. Four stars.

GREENFIELD, ROBERT ARTHUR
Entered service May 15, 1917. Capt., E. O. R. C., June, 1917; Maj., Engrs., N. A., July, 1918. With 303d Engrs.

GREGORY, JOHN HERBERT
Entered service Nov. 8, 1918; Capt., San. C., U. S. A., Nov. 2, 1918. Discharged Dec. 20, 1918. Camp Greenleaf.

GREGORY, LUTHER ELWOOD
Entered service Apr. 5, 1898; through all grades in C. E. C., U. S. N., to Capt., July 1, 1917. Public Works Officer, Puget Sound Navy Yard and Boston Navy Yard.

GREGORY, WILLIAM BENJAMIN
Entered service Sept. 26, 1917; Maj., E. O. R. C., March 10, 1917. Overseas service Oct. 3, 1917–Jan. 30, 1919. Discharged Feb. 6, 1919. Water Supply Office, H. Q., Div. Constr. and Forestry, A. E. F. Citation from Gen. Pershing for meritorious service.

GREHAN, BERNARD HENRY
Entered service May 12, 1917; 2d Lt., E. O. R. C., June 28, 1917; 1st Lt., Engr. R. C., Dec. 31, 1917. Overseas service Aug. 25, 1918–July 6, 1919. Discharged July 25, 1919. With 312th Engrs.

GRIEST, MAURICE
Entered service Sept. 22, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 27, 1918. With R. T. C., Washington, D. C.; 4th Engr. Training Regt., Camp Humphreys.

GRIFFIN, JOHN ALDEN
Entered service May 8, 1917; Capt., Engrs., N. A., Aug. 23, 1917; Maj., Engrs., U. S. A., Oct., 1918. Overseas service July 6, 1918–Sept. 11, 1918. Discharged Jan. 25, 1919. With 316th and 607th Engrs.

GRIGSBY, WALTER BERTON
Entered service Sept. 4, 1917; 2d Lt., E. O. R. C., July 2, 1917; 1st Lt., Engrs., N. A., Dec. 11, 1917. Discharged Dec. 18, 1919. With 305th Engrs.; with 9th Engrs. as Adj., Supply Officer and Topographic Officer.

GRIMES, JAMES EDWARD
Entered service Mar. 23, 1918, as Capt., Q. M. C., Constr. Div., N. A. Discharged Mar. 28, 1919. Asst. to Const. Q. M., Aberdeen Proving Grounds and T. A. Gillespie Loading Co., Morgan, N. J.; Asst. to Officer in Chg. of Utilities, Camp Grant.

GRINDROD, ERVIN SUTTON
Entered service Sept. 28, 1917; 1st Lt., Engrs., N. A., Aug. 15, 1917; Capt., Engrs., U. S. A., May 7, 1919. Overseas service May 10, 1918–July 5, 1919. Discharged July 29, 1919. With 103d Engrs.; Bn. Adj. and Supply Officer, 33d Engrs.; Engr. Officer in chg. constr., Div. of Constr. and Forestry, A. E. F. Citation from Gen. Pershing for meritorious service.

GRISWOLD, HARRY TODD
Entered service July 28, 1917, as 2d Lt., C. A. C., N. A.; 1st Lt., C. A. C., N. A., Nov., 1917. Overseas service Mar. 28, 1918–Jan. 28, 1919. Discharged Jan. 28, 1919. With H. Q. Co., 56th Regt., C. A. C.; Artillery Intelligence Officer, G-2, 1st Army, A. E. F.

GRISWOLD, HECTOR CLINTON
Entered service June 15, 1917 as Lt., Jr. Grade, C. E. C., U. S. N.; Lt., C. E. C., U. S. N., Oct. 15, 1917. In chg. constr. at Washington, D. C.; member Examining Bd.; in chg. constr. at naval ammunition depots in New York Dist., and Public Works Office, Fleet Supply Base, South Brooklyn, N. Y.

GRODSKE, WALTER JOHN
Capt., Engrs., U. S. A.*

GROSS, FREDERICK HENRY
Entered service May 10, 1917, as 2d Lt., E. O. R. C.; 1st Lt., Engrs., U. S. A., Sept., 1918. Overseas service May 24, 1918–June 23, 1919. Discharged July 3, 1919. With 303d and 113th Engrs.

GROSS, HENRY McCORMICK
Entered service June, 1917, as 1st Lt., Inf.; 1st Lt., Machine Gun Bn., Aug. 25, 1917; Capt., Inf., U. S. A., Oct., 1918. Overseas service Oct. 1917–Apr. 1919. Discharged May, 1919. With 8th Penna. Inf.; 149th Machine Gun Bn.; Chf. Machine Gun Instr., 92d Div.; Aide to Maj.-Gen. William H. Hay. Cited in Brig. Operations Report, 55th Inf. Brig. One star.

GRUNAUER, MORTIMER
Entered service Apr. 4, 1918, as Pvt., F. A., N. A.; Sgt., F. A., N. A., Aug. 1, 1918. Overseas service Apr. 24, 1918–July 18, 1919. Discharged July 23, 1919. With 304th F. A.; Instr. in Orientation, F. A. School, Camp de Souge, France. One star.

GUILLEMETTE, JOSEPH DYDIME
Entered service Apr. 11, 1918, as Pvt., 1st Class, Aviation Section, Sig. C., N. A.; 2d Lt., A. S., N. A., July 19, 1918. Overseas service July 30, 1918–Jan. 7, 1919. Discharged Jan. 7, 1919. Office, Chf. Sig. Officer, H. Q., S. O. S., A. E. F.

GUNTHER, HERMAN DIETRICH
Entered service Apr. 2, 1918, as Pvt., Inf., N. A. Overseas service June 23, 1918–Mar. 23, 1919. Discharged Apr. 7, 1919. With 147th Inf. Two stars.

GUPPY, BENJAMIN WILDER
Entered service May 16, 1917, as Maj., E. O. R. C.; Lt. Col., Engrs., N. A., Mar. 19, 1918. Overseas service July 27, 1917–Feb. 9, 1919. Discharged Feb. 11, 1919. C. O., 14th Engrs.; 2d in command, Camp Hunt; Supt., Port Terminal Facilities, St. Nazaire, France; British front near Calais; Marne Salient; with 1st Army, 3d Corps, A. E. F. Three stars.

GUPTILL, JOSEPH RICKER
Entered service Nov. 3, 1917; Pvt., Engrs., N. A., Nov. 14, 1917. Overseas service, Mar. 31, 1918–June 16, 1919. Discharged June 27, 1919. With 23d Engrs., gen. constr. highways and railroads; Meuse-Argonne offensive. One star.

GUTMAN, DAVID
Entered service Jan. 4, 1918; Maj., Engrs., N. A., Sept. 17, 1917. Discharged Dec. 28, 1918. E. O. T. C., Camp Lee; Constr. Div., Washington, D. C.; Bn. Comdr., 4th Engr. Training Regt., and in chg. of constr., Camp Humphreys; with 5th Prov. Bn., in chg. of constr., Camp Benjamin Harrison.

HAAS, PHILIP LIPPMAN
Capt., Engrs., U. S. A.*

HADLEY, ALLEN STACY
Entered service Sept. 5, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 10, 1919. With 139th and 218th Engrs.; detailed to M. T. C.

HAGUE, WILLIAM
Entered service 1917 as 1st Lt., Engr. R. C. With 116th Engrs., at the Presidio, Vancouver Barracks, Camps American Lake, Charlotte, Mineola, and in France. Died Jan. 2, 1918, in active service in France.

HAINS, PETER CONOVER
Entered service June 1, 1857; through various grades to Maj. Gen., U. S. A. (*Retired*). Assigned to active duty Sept. 19, 1917. In chg., River and Harbor Works and Defenses, Norfolk, Va., and Chesapeake Bay.

HALCOMBE, NORMAN MARSHALL
Entered service May 1, 1915, as 2d Lt., Royal Engrs., British Army; Lt., Royal Engrs., June, 1915; Capt., Royal Engrs., Aug. 4, 1915; Capt., Royal Flying C., Nov. 1, 1916; Maj., Royal Flying C., Nov. 16, 1917. Overseas service, 1915–1919. Co. Comdr., 31st Cheshire Field Co., R. E., 21st Welsh Field Co., R. E., and 21st Cheshire Field Co., R. E. (Gallipoli); special evacuation work in Egypt; special transport work in Western Desert, Egypt and Sinai; organized and developed Engine Repair Depots, Royal Flying Corps, in Egypt and Palestine; with Technical Dept., Parachute Sec., Aircraft Production, London, England; Staff Officer, Indian Air Force, India. Mentioned three times in dispatches. Died Feb. 15, 1919, in active service at Port Said, Egypt.

HALDEMAN, WALTER STANLEY
Entered service June 11, 1917, as Capt., E. O. R. C. Discharged Dec. 15, 1918. Acting Bn. Adj., 314th Engrs.; Instr., E. O. T. S., Camp Funston; Co. Comdr., 3d Engrs. Training Regt.; Bn. Adj., 606th Engrs.

HALE, RICHARD KING
Entered service July 25, 1917, as Lt. Col., F. A., N. A.; Col., F. A., N. A., Nov. 7, 1917. Overseas service Sept. 9, 1917–Apr. 4, 1919. Discharged June 14, 1919. C. O., 103d F. A.; with 101st F. A.; G. S. College overseas; Asst. Chf. of Staff (G-Q), 2d Army Corps; Gen. Staff 2d Army Corps with British Army; Asst. Chf. of Staff, American Embarkation Center; Chf. of Staff, 26th Div.

HALL, BENJAMIN MORTIMER, JR.
Entered service May 14, 1917; 1st Lt. Engrs., N. A., Aug. 1, 1917; Capt., Engrs., U. S. A., Nov. 6, 1918. Overseas service Oct. 31, 1917–June 17, 1919. Discharged June 28, 1919. Asst. to Officer in Chg. Water Supply, Base Sec. No. 1, A. E. F.; Co. Comdr., 26th Engrs., during St. Mihiel offensive; Staff of Chf. Engr., 2d Army, as Asst. Water Supply Officer; Acting Officer in Chg. Water Supply Sec., Office of Chf. Engr., A. E. F. One star.

HALL, CHARLES LACEY
Through all grades in C. of E., U. S. A., to Maj., May 15, 1917; Lt. Col., Dec. 20, 1917; Col. (temporary), Aug. 1, 1918. Overseas service July 28, 1917–June 28, 1919. Reverted to permanent grade Oct. 6, 1919. Adj., 12th Engrs.; G. S., A. E. F.; Asst. Sec. Engr., Advance Sec., A. E. F. Cited by Comdr. in Chf. for services as Chf., G-2-c, 1st Army, and in connection with organization of sound and flash ranging troops. Four stars.

HALL, GILBERT G.
Entered service Sept. 2, 1917; 1st Lt., Engrs., N. A., Dec. 6, 1917. Overseas service, Jan. 2, 1918–July 29, 1919. Discharged Aug. 16, 1919. With 20th Engrs. as Ry. Transfer Officer, and Camp Comdr., "Burned Area" Dist., Pontenx-les-Forges, France.

HALL, JULIUS REED
Entered service Sept. 25, 1917, as 1st Lt., E. O. R. C.; Capt., Engrs., U. S. A., May 12, 1919. Overseas service May 5, 1918–July 12, 1919. Discharged Aug. 6, 1919. With 515th Engrs. on Depot Constr., Gievres, France.

HALL, LOUIS WELLS
Entered service July 19, 1917, as Capt. E. O. R. C. Overseas service July 23, 1917–Feb. 11, 1919. Discharged Feb. 17, 1919. Assigned to engrs. in 1st Div., preparing camps, hosps., etc.; Office, Chf. Engr., A. E. F., in chg. Drafting and Map Production Dept., Div Constr. and Forestry.

HALL, OLIVER ANTRUM
Entered service Nov. 26, 1917; Capt., Engrs., N. A., Dec. 6, 1917. Overseas service Mar. 28, 1918–July 27, 1919. With 23d Engrs.; Engr. Officer in chg. design and constr. ord. repair shops and bldgs. at Clermont-Ferrand, France; with regt. at front as Liaison Officer between 1st, 3d and 5th Corps, and H. Q., 1st Army Roads Office; assigned to C. W. S.; Instr. C. W. S. school at Lakehurst Proving Grounds, N. J. Two stars.

HALL, WARREN ESTERLY
Entered service Sept. 25, 1917; Capt., Engrs., N. A., Nov. 27, 1917. Overseas service Jan. 4, 1918–Mar. 25, 1919. Discharged July 18, 1919. With 506th Engrs.; on heavy railroad constr., Base Sec. No. 2, A. E. F.; Asst. Water Supply Engr. and Water Supply Engr., Base Sec. No. 2, A. E. F. One star.

HALLIHAN, JOHN PHILIP
Entered service Aug. 3, 1918; Maj., Engrs., U. S. A., Aug. 17, 1918. Overseas service Oct. 17, 1918–Mar. 6, 1919. Discharged Mar. 12, 1919. C. O., 70th Engrs. and Post Comdr., Ft. Douglas; in Engr. Supply Sec., Div. Military Eng. and Engr. Supplies, A. E. F.; in Transportation Sec. Engr. Dept., American Commission to Negotiate Peace.

HALSEMA, EUSEBIUS JULIUS
Capt., Engrs., U. S. A.*

HAMILTON, EDWARD PARMELEE
Entered service Aug. 15, 1917, as 2d Lt., F. A., N. A.; 1st Lt., F. A., N. A., Jan. 2, 1918. Overseas service Apr. 24, 1918–Apr. 29, 1919. Discharged May 10, 1919. With 306th F. A. as Battery Officer, Regimental Reconnaissance and Personnel Adj.; Baccarat and Vesle Sectors, Oise-Aisne and Meuse-Argonne offensives. Two stars. Prisoner of war, Sept. 28 to Nov. 29, 1918.

HAMMILL, HAROLD BERNARD
Entered service Apr. 15, 1918, as Lt., Jr. Grade, C. E. C., U. S. N. R. F. Overseas service May 16, 1918–Feb. 1, 1919. Released from active duty Mar. 17, 1919. With unit on constr. of high-power radio plant at Croix d'Huis, France.

HAMMOND, JOHN MILLER
Entered service May 16, 1918, as Capt., Q. M. C., Constr. Div., N. A. On constr. projects at New Orleans Army Supply Base, Rock Island Arsenal and Honolulu, Hawaii; Const. Q. M. at Schofield Barracks, Hawaii.

HANAVAN, WILLIAM LAWRENCE
Entered service May 8, 1917; 1st Lt., Inf., N. G., Nov. 6, 1916; Capt., Inf., N. A., July 12, 1918. Overseas service Sept. 8, 1917–Jan. 5, 1919. Discharged May 5, 1919. Inf. Officers' School, France; Co. Comdr. and Bn. Adj., 9th Inf.; Machine Gun School, Gondrecourt, France; Personnel Adj. and Comdr. railhead detachment, 9th Inf.

HANCOCK, HENRY SYDNEY, JR.
Entered service Dec. 9, 1915, British Army; 2d Lt. Royal Engrs., June 25, 1916; Lt., Royal Engrs., Apr. 18, 1917. Overseas service Oct. 28, 1915–Oct. 6, 1919. Discharged Oct. 6, 1919. With 254th Tunneling Co.; Field Engr., 1st Corps, in chg. support lines and corps defenses, heavy gun emplacements, water supplies, encampments, etc. (Acting Maj.); with 170th Tunneling Co. Wounds—"None sufficiently serious to prevent my carrying on."

HANIQUE, JULES EDMOND
Entered service May 10, 1917; Capt., E. O. R. C., June 20, 1917; Maj., Engr. R. C., May 22, 1918. Overseas service July 13, 1918–Aug. 14, 1919. Discharged Aug. 22, 1919. Adj., 316th Engrs., Camp Lewis; with 537th Engrs.; Acting Corps Road Engr. during Meuse-Argonne offensive; Liaison Officer, French Staff, Verdun; Dist. Road Engr. Citation for efficiency during Argonne drive. Two stars.

HANNAN, DAVID EDWARD
Capt., Engrs., U. S. A., A. E. F.*

HANSEN, PAUL
Entered service May 7, 1917; Capt., E. O. R. C., May 1, 1917. Overseas service Aug. 5, 1917–Apr. 7, 1919. Discharged Apr. 15, 1919. On staff duty at G. H. Q., A. E. F.; H. Q., Advance Sec., A. E. F.; with 1st and 2d Armies and with Peace Commission.

HAPGOOD, FREDERIC HERBERT
Entered service Sept. 29, 1917, as Pvt., Engrs., N. A.; Cpl., Engrs., N. A., Oct. 23, 1917; M. E., Jr. Grade, Engrs., N. A., Mar. 8, 1918; M. E., Sr. Grade, Engrs., U. S. A., Jan. 2, 1919. Overseas service Oct. 31, 1917–Mar. 12, 1919. Discharged Mar. 21, 1919. With Water Supply Sec., Office of Chf. Engr., A. E. F.; Sec. Engr., Advance Sec. A. E. F.; Chf. Engr., 1st Army at Vraincourt, Romagne-sous-Montfaucon, Dun-sur-Meuse; H. Q., 2d Bn., 26th Engrs.

HARDAWAY, BENJAMIN HURT, JR.
Maj., Inf., U. S. A.*

HARDING, CHESTER
Entered service June 14, 1885; through all grades in C. of E., U. S. A., to Col., May 15, 1917. Governor, Panama Canal, Jan. 11, 1917. Retired from active service as Brig. Gen., U. S. A., Apr. 1, 1920.

HARDING, RALPH LYMAN
Entered service Sept. 2, 1917; Capt., E. O. R. C., July 26, 1917; Major, Engrs., U. S. A., Nov. 4, 1918. Overseas service May 12, 1918–June 9, 1919. Discharged Oct. 27, 1919. Adj. and Bn. Comdr., 32d Engrs.

HARPS, HARRY MACY
Capt., Engrs., U. S. A.*

HARRAH, ORIN WILSON
Entered service Sept. 2, 1917; Capt., Engrs., N. A., Jan., 1918. Discharged Mar. 28, 1919. Co. Comdr., 319th Engrs., Camps Fremont and Humphreys; Co. Comdr., 219th Engrs., Camps Dodge and Humphreys.

HARRINGTON, ARTHUR WILLIAM
Entered service Nov. 6, 1918, as 1st Lt., San. C., U. S. A. Discharged July 16, 1920. Camp San. Engr. at Eberts Field, Barron Field, Ellington Field, Camp Benning, Camp Humphreys; Asst. Const. Q. M., Camp Bragg.

HARRIS, FRANK SAMPSON MASON
Entered service Mar. 15, 1917, as Lt., U. S. N. R. F. Released from active service Mar. 15, 1919. On U. S. S. *Pueblo*, scout duty in South Atlantic, and convoy duty, North Atlantic; Bureau of Steam Eng., Navy Dept., Washington, D. C.

HARRIS, FREDERIC ROBERT
Entered service Jan. 3, 1903; through all grades in C. E. C., U. S. N., to Rear Admiral, Aug., 1916. Chf., Bureau of Yards and Docks; Gen. Mgr., Shipping Bd., Emergency Fleet Corp.; Aide, Public Works, 5th Naval Dist.; Pres. of Bd. to Control War Activities. Navy Cross.

HARRISON, CHRISTOPHER
1st Lt., Inf., U. S. A.*

HARRISON, WILLIAM BURR
Entered service Oct. 6, 1917; Maj., E. O. R. C., Feb. 19, 1917. Discharged Dec. 20, 1918. Office, Chf. of Engrs., special engr. equipment, tools, etc.; with 153d Engrs.

HART, SAMUEL ALEXANDER
Entered service Sept. 2, 1917, as 1st Lt., Engrs., N. A. Overseas service Mar. 30, 1918–June 9, 1919. Discharged June 17, 1919. With 23d Engrs.; S. O. S., A. E. F.; St Mihiel and Meuse-Argonne offensives. Two stars.

HARTER, ALOYSIUS FRANK
Entered service Sept. 27, 1917; Capt., Engrs., N. A., May 1, 1918. Overseas service June 22, 1918–Sept. 25, 1919. Discharged Oct. 15, 1919. Co. Comdr., 527th Engrs.; in chg. camp constr. at Montoir, France; with 2d French and 1st American Armies; St. Mihiel and Meuse-Argonne offensives. Two stars.

HARTS, WILLIAM WRIGHT
Entered service Aug. 28, 1885; through all grades in C. of E., U. S. A., to Col., June 23, 1917; Brig. Gen., U. S. A., Dec. 17, 1917. Overseas service Dec. 5, 1917–Mar. 30, 1920. C. O., 6th Engrs.; preparation American training areas in France; faculty, Gen. Staff School, France; with 51st British Div. at Ypres; with 2d British Div., Cambrai; with 16th French Div., Champagne; C. O., American troops with British except 2d Corps; C. O., Dist. of Paris; Chf. of Staff, Third Army. Distinguished Service Medal; Commandeur, Legion d'Honneur; Knight Commander, Order of St. Michael and St. George, Great Britain; Commander, Order of the Crown, Belgium; Officier, Order du Denile II, Montenegro.

HARVEY, ALDEN WALES
Entered service Sept. 2, 1917; 2d Lt., E. O. R. C., July 2, 1917; 1st Lt., Engrs., U. S. A., Apr. 9, 1919. Overseas service Mar. 22, 1918–June 9, 1919. Discharged July 2, 1919. Co. Comdr, 511th Engrs., constr. gas mask factory, etc.

HASBROUCK, OSCAR
Entered service Dec. 28, 1917, as Capt., Engr., R. C. Overseas service Aug. 28, 1918–Aug. 1, 1919. Discharged Aug. 1, 1919. Co. Comdr. and C. O., 522d Engrs.; commanded sector of 2d line of defense; attached to 2d Army. Two stars.

HASKINS, CHARLES ARTHUR
Entered service Dec. 27, 1917, as Capt., San. C., N. A. Discharged Aug. 6, 1919. Asst. to Camp San. Insp., Camp Sherman; Camp San. Engr., Camp Devens; Special Insp. and Officer in Chg. San. Eng. Sec., Surgeon General's Office, Washington, D. C.; wrote San. Eng. Sec. of "Medical History of the War."

HASKINS, JOHN CHRISTOPHER
Entered service Aug. 3, 1918, as 1st Lt., Engrs., N. A. Overseas service Aug., 1918–July, 1919. Discharged July 23, 1919. Co. Comdr., 22d Engrs., and Supply Officer in chg. of constr. work, light rys., roads and bldgs.

HASTINGS, RUSSELL PLATT
Entered service Dec. 28, 1917; 1st Lt., Engrs., N. A., Feb. 8, 1918. Overseas service July, 1918–Feb., 1919. Discharged Mar., 1919. With 26th Engrs.

HASWELL, JOHN ROBERT
Entered service Sept. 2, 1917, as Capt., E. O. R. C. Overseas service Jan. 24, 1918–Mar. 6, 1919. Discharged Mar. 18, 1919. With 23d Engrs., Camp Meade; 1st Engrs., Montdidier-Noyon defensive; Meuse-Argonne offensive, Engrs., First Army Corps, A. E. F.; Co. Comdr., 317th Engrs. Two stars.

HATCH, FREDERICK NATHANIEL
Entered service May 14, 1917; Capt., E. O. R. C., July 5, 1917. Overseas service Dec. 12, 1917–July 29, 1919. Discharged Aug. 16, 1919. With 312th and 35th Engrs.; 96th Co., T. C.; special duty H. Q., Base Sec. No. 2, A. E. F.

HAUSER, KENNETH DOUGLAS
Entered service Apr., 1917; Capt., E. O. R. C., June 19, 1917; Maj., Engrs., U. S. A., Oct. 3, 1918. Overseas service Aug. 9, 1917–Apr. 28, 1919. Discharged May 29, 1919. Co. Comdr., 18th Engrs. on railway and dock constr., Base Sec. No. 2, A. E. F.; Asst. Sec. Engr.; Chf. Engr. in converting Pauillac Naval Air Sta. into Army Embarkation Camp.

HAWES, GEORGE RAYMOND
Entered service June 1, 1918, as Pvt., Engrs., N. A.; Sgt., Engrs., U. S. A., Dec. 1918; M. E., Sr. Grade, Engrs., U. S. A., May, 1919. Overseas service Oct. 20, 1918–July 16, 1919. Discharged July 29, 1919. With 472d and 137th Engrs.; attached to Sec. Engr. Office, Base Sec. No. 1, A. E. F.

HAWLEY, JOHN BLACKSTOCK
Entered service July 7, 1917; Maj., E. O. R. C., May 16, 1917. Overseas service, Nov. 26, 1917–Mar. 13, 1919. Discharged Apr. 18, 1919. With 503d Engrs.; Engr. Officer in chg. water supply and sanitation, Base Sec. No. 1, A. E. F. Officier du l'Academie, with Univ. Palms.

HAWLEY, ROBINSON WILBER
Entered service Oct. 15, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 5, 1918. With E. O. T. S.

HAYDOCK, CHARLES
Pvt., E. O. T. S., U. S. A.*

HAYES, FERDINAND EUGENE, JR.
Entered service July 29, 1918, as Chf. Carpenter's Mate, U. S. N., R. F.; Lt., Jr. Grade, C. E. C., U. S. N. R. F., Mar. 4, 1919.

HAYNE, DANIEL CARLOS
Entered service June 13, 1917, as Capt., E. O. R. C. Discharged Dec. 8, 1917. (With Emergency Fleet Corp., Hog Island Shipyard.)

HAYS, DONALD SYMINGTON
Entered service Nov. 23, 1917; Capt., Engrs., N. A., Nov. 9, 1917. Discharged Oct. 29, 1919. In Gen. Engr. Depot as Purchasing Officer contractors equipment and prime movers. Invented two military devices.

HAYS, JOHN COFFEE
Entered service Oct. 25, 1917, as Maj., Q. M. C., N. A. Discharged Oct. 14, 1919. Const. Q. M. and Camp Utilities Officer at Camp Lewis; Special assignments supervising utilities, Camps Kearny and Fremont.

HAYT, ROBERT OLCOTT
1st Lt., Engrs., U. S. A.*

HAZELTON, WILLIAM SYLVESTER
Entered service Nov. 8, 1918, as Capt., Ord. Dept., U. S. A. Discharged Aug. 14, 1919. Production Div., Detroit Dist., in chg. Salvage Bd.

HAZLEHURST, JAMES NISBET
Entered service June 20, 1917, as Maj., Engrs., U. S. A. Overseas service, 1917–1919. Staff Officer in chg. of Water Supply Sec., Dept. of the Southeast, U. S. A.; Prin. Asst., Water Supply Sec., A. E. F., Chaumont, France; attached to Third British Army as Observer, Valley of the Somme; Prin. Asst., Water Supply Sec., A. E. F., Tours, France; Engr. Officer in chg. of constr. and Water Supply Officer, Army Base No. 6, A. E. F.; Member, Am. Peace Comm. with Economical and Financial Sec., Belgian Comm., as Director of Div. of Damages to Bldgs., Brussels, Belgium. Died Feb. 9, 1919, in service at Brussels, Belgium.

HEALEY, CHARLES FRANK
Entered service July 17, 1917, as Capt., Engrs., N. A. Overseas service Jan. 27, 1918–Jan. 13, 1919. Discharged Jan. 17, 1919. Dist. Engr., T. C., with H. Q. at St. Nazaire, France; in chg. constr. railroad docks and warehouses, Base Secs. No. 1, 2 and 5.

HEBARD, ROY WILLIAM
Entered service May 15, 1918, as Capt., Engrs., U. S. A.; Maj., Engrs., U. S. A., Aug. 1, 1918. Overseas service Sept. 14, 1918–June 25, 1919. Discharged July 20, 1919. Bn. Comdr. and C. O. 22d Engrs., St. Mihiel Sector. One star.

HECK, NICHOLAS HUNTER
Entered service Sept. 24, 1917, as Lt., U. S. N. R. F.; Lt. Comdr., U. S. N. R. F., Jan. 19, 1919. Overseas service Sept. 24, 1918–Dec. 18, 1918. Released from active service Mar. 19, 1919. At Naval Experimental Station, New London, Conn., on development of a number of submarine devices; attached to Naval H. Q., London.

HEDGES, SAMUEL HAMILTON
Entered service Jan. 23, 1917, as Maj., E. O. R. C. Resigned to enter civilian war work Jan. 8, 1918.

HEED, SAMUEL DARLINGTON
Entered service June 29, 1918; Capt., Ord. Dept., N. A., July 1, 1918; Maj., Ord. Dept., U. S. A., Aug. 2, 1919. Discharged Feb. 4, 1920. Asst. Engr. in Plant Facilities Branch, Production Div.; Officer in Chg., U. S. Propellant Plant, Tullytown, Pa.; Phila. Dist., Ord. Claims Bd.

HENDRICKS, ERNEST DEMAREST
Entered service May 8, 1917, as 1st Lt., E. O. R. C.; Capt., E. O. R. C., Aug. 16, 1917. Overseas service May 26, 1918–Apr. 27, 1919. Discharged May 19, 1919. Co. Comdr., 303d Engrs. Citation from Gen. Pershing for gallantry in action at Grand-Pre, France.

HENDRIE, JOHN GIBSON†
Capt., Engrs., U. S. A.*

HENNING, CHARLES SUMNER, JR.
Entered service Sept. 10, 1917, as Pvt., Engrs., N. A.; 2d Lt., Engrs., U. S. A., Sept. 6, 1918; 1st Lt., Engrs., U. S. A., May 9, 1919. Overseas service Dec. 26, 1917–June 9, 1919. Discharged July 9, 1919. With 21st Engrs., light ry. constr.

HENRY, DAVID EDWARD
Entered service Dec. 28, 1917; Capt., E. O. R. C., July, 1917. Discharged Apr. 12, 1918. Officers Training Camp.

HENRY, EARLE UNDERWOOD
Entered service June 27, 1917, as 1st Lt., E. O. R. C.; Capt., Engrs., U. S. A., Dec., 1918. Overseas service Jan. 6, 1918–Feb. 9, 1919. Discharged Feb. 12, 1919. Assigned to T. C.

HERINGTON, GEORGE B.
Entered service Sept. 18, 1918, as Capt., A. S. A. P., Spruce Production Div.; Maj., A. S. A. P., Spruce Production Div., Oct. 1, 1918. Discharged Mar. 13, 1919. Supervisor, Coos Bay Production Dist.; Mgr., Spruce Production Rys.; Mgr., Spruce Production Corp.; Pres., Claims Bd., involving settlement of large contracts, selling lumber, etc.

HERKNESS, LINDSAY COATES
Entered service June 15, 1905; through all grades in C. of E., U. S. A., to Lt. Col., Feb. 28, 1918; Col. Engrs., U. S. A., Aug. 20, 1918. Overseas service Mar. 25, 1918–Sept. 20, 1918. Discharged Sept. 27, 1919. In chg. coast and aerial defense, New York City; 2d in command, 302d Engrs.; C. O., Camp Leach.

HERSHEY, JOHN LOGAN
Entered service Sept. 16, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 4, 1918. Training Camp and 4th Engr. Training Regt.

HERZIG, SOLON
Entered service Sept. 25, 1917; 1st Lt., Engrs., N. A., Aug. 16, 1917; Capt., Engrs., U. S. A., May 6, 1919. Overseas service Feb. 27, 1918–May 23, 1919. Discharged June 16, 1919. With 307th Engrs.; with 25th Engrs., Co. Comdr., St. Sulpice, France; Engr. Officer in chg. constr., Rochefort, France; Meuse-Argonne operations. One star.

HESLOP, DERWENT GORDON
Entered service Dec. 6, 1915, British Army, as Sapper, Royal Engrs.; through all grades to Lt., Nov. 27, 1916; Capt., Jan. 1, 1918; Maj., Feb. 6, 1919. Overseas service Feb. 2, 1916–May 18, 1920. Discharged May 30, 1920. C. O., Light Ry. Surveys, Palestine; Chf. Engr., Hedjaz and Damascus Beyrout Ry.; Chf. Engr., Bagdad Ry.; C. O., Haita-Bagdad Ry. Survey. Croix de Guerre; mentioned in despatches by Field Marshal Lord Allenby.

HESLOP, PAUL LOVERIDGE
Entered service Sept. 10, 1918, as Pvt., Inf., U. S. A.; Pvt., Ord. Dept., U. S. A., Nov. 7, 1918. Discharged Jan. 3, 1919. Inf., Camp Sherman; Army Ord. School, Carnegie Inst. Technology.

HEUER, WILLIAM HENRY
Entered service as Cadet, 1861; retired as Col., C. of E., U. S. A., Mar. 2, 1907; on active list as Col., C. of E., U. S. A., Apr. 26, 1917. Retired again Aug. 1, 1919. In chg. rivers and harbors work in California and fortification work in San Francisco Bay; Div. Engr., Pacific Div.; member Bd. for establishing harbor lines in and near San Francisco.

HEWETT, MAURICE WILLIAM
Entered service May 12, 1917; 2d Lt., E. O. R. C., June, 1917; 1st Lt., E. O. R. C., Nov. 1, 1917. Overseas service Jan., 1918–May, 1919. Discharged June 19, 1919. With 508th Engrs. at Bazoilles-sur-Meuse, Vosges, hospital constr.; road constr. at La Rochelle, France.

HEWS, WELLINGTON PRESCOTT
Entered service Dec. 27, 1917, as Capt., E. O. R. C. Overseas service, July 13, 1918–Sept. 24, 1919. Discharged Sept. 27, 1919. Co. Comdr. 62d Engrs., Ft. Benjamin Harrison; C. O., T. C., Casual Camp, Poincon; C. O., T. C., Is-sur-Tille, France.

† Died June 18, 1919.

HICKOK, CHARLES WILLIAM
Entered service Apr. 16, 1918, as Pvt., Aviation Sec., Sig. C., N. A.; Sgt., Aviation Sec., Sig. C., U. S. A., Jan. 10, 1919; Sgt., 1st Class, Aviation Sec., Sig. C., U. S. A., Feb. 10, 1919. Discharged June 14, 1919. School of meteorology and aerology, College Sta. Tex.; in chg. Aerological Sta., Ellendale, N. D., Meteorological Sta. at Love Field and at Ellington Field.

HICKOK, CLIFTON EWING
Entered service May 1, 1918, as Capt., Engrs., N. A. Discharged Jan. 1, 1919. On staff of Chf. of Engrs., Washington, D. C.

HIGGINS, CHARLES HOUCHIN
Entered service Aug. 21, 1918, as Maj., Ord. Dept., U. S. A. Discharged Dec. 24, 1918. Production Div. and Supply Div.; training for Div. Ord. Officer.

HILDER, FRAZER CROSWELL
Entered service May 8, 1917; Capt., E. O. R. C., Jan. 23, 1917; Maj., Engrs., U. S. A., Apr. 10, 1919. Overseas service Dec. 19, 1917–Aug. 19, 1919. Discharged Oct. 31, 1919. C. O., 13th Bn., Depot Brig., Camp Lee; Regtl. Adj., 317th Engrs.; Asst. to Constr. Officer and Constr. Officer, Base Sec. No. 3, A. E. F.

HILES, ELMER KIRKPATRICK
Entered service May 20, 1917; Capt., E. O. R. C., Feb. 25, 1917; Maj., E. O. R. C., Aug. 10, 1917; Lt. Col., Engrs., U. S. A., Apr. 7, 1919. Overseas service July 9, 1917–June 6, 1919. Discharged July 3, 1919. In chg. ry. constr., Issoudun, France; in chg. depot and sta. constr., Mehun and Liffot-le-Grand, France; in chg. Contract Sec., Eng. Purchasing Office, Paris; in chg. settlement Engr. C. Contracts, A. E. F.

HILTON, HARRY LeGRAND
Entered service Apr. 11, 1917, as Lt., Jr. Grade, C. E. C., U. S. N. R. F.; Lt., C. E. C., U. S. N. R. F., July 24, 1918. Special duties at Portsmouth, N. H., Navy Yard, with radio service at Bar Harbor, Me.

HINRICHS, ADOLF
1st Lt., Engrs., U. S. A.*

HIRZEL, ALFRED SPARKS
Entered service Mar., 1916, as Capt., Inf., Del. N. G.; Capt., Inf., N. A., 1918; Maj., Inf. U. S. A., Mar. 1919. Overseas service Aug., 1918–July 5, 1919. Discharged July 11, 1919. With 59th Pioneer Inf. and 2d Colonials, French Army, road work at St. Mihiel. Two stars.

HJORTH, LAWRENCE RASMUS
Entered service Dec. 7, 1917, as Pvt., Sig. C., N. A.; Pvt., Ord. Dept., N. A., Mar. 4, 1918; 2d Lt., Ord. Dept., U. S. A., Oct. 17, 1918. Overseas service Oct. 27, 1918–May 5, 1919. Discharged May 25, 1919. With 302d Field Sig. Bn.; supervised handling explosives and ammunitions, Raritan Arsenal; Base Ord. Depot, Mehun, France.

HOAD, WILLIAM CHRISTIAN
Entered service Jan. 29, 1918, as Maj., San. C., N. A.; Lt. Col., San. C., U. S. A., Aug. 16, 1918. Discharged Feb. 15, 1919. Director School for San. Engrs. and Camp San. Engr., Camp Greenleaf; Advisory Engr. to Surgeon Gen., Washington, D. C.

HOAR, ALLEN
Entered service May 14, 1917; Lt., Jr. Grade, C. E. C., U. S. N., Mar. 17, 1917; Lt., C. E. C., U. S. N., May 1, 1918. Overseas service Nov. 10, 1918–July 19, 1919. Asst. Public Works Officer and Acting Public Works Officer, Mare Island Navy Yard; Officer in Chg. radio expedition to Siberia.

HOBBS, HENRY WEBSTER
Entered service July 16, 1918, as Maj., Engrs., U. S. A.; Lt. Col., Engrs., U. S. A., Sept. 11, 1919. Discharged Oct. 31, 1919. In office Chf. of Engrs., Asst. to Officer in Chg. and Officer in Chg. all seacoast fortification construction.

HODGE, HENRY WILSON †
Entered service July 19, 1917, as Maj., Engrs., N. A.; Lt. Col., Engrs., N. A., Oct. 26, 1917; Col., Engrs., U. S. A., Aug. 13, 1918. Overseas service, July 29, 1917–Jan. 18, 1919. Discharged Jan. 22, 1919. In chg. of design of docks and railroad structures; Mgr. of Roads, and Asst. Chf. Engr. in chg. of military bridges, A. E. F. Cited by Gen. Pershing for exceptionally meritorious and conspicuous service.

HODGES, HARRY FOOTE
Entered service July 1, 1877, as cadet, U. S. Military Academy; Maj. Gen., N. A., Aug. 5, 1917. Overseas service Dec. 1, 1917–Feb. 1, 1918 and July 5, 1918–Dec. 22, 1918. Returned to rank of Brig. Gen., U. S. A., June 30, 1920. C. O., 76th Div.

HOFFMAN, EUGENE ROBERT
Entered service Feb. 9, 1918, as Pvt., Engrs., N. A.; M. E., Sr. Grade, Engrs. Overseas service Mar. 24, 1918–Mar. 24, 1919. Discharged May 2, 1919. With 26th Engrs.

HOGAN, JOHN PHILIP
Entered service May 2, 1917; Capt., E. O. R. C., Jan. 23, 1917; Maj., G. S., U. S. A., Oct. 1, 1918; Lt. Col., G. S., U. S. A., May 1, 1919. Overseas service July 14, 1917–July 5, 1919. Discharged July 11, 1919. Supply Officer and Adj., 11th Engrs.; Executive Officer, G-2-C, G. H. Q., A. E. F.; attached to G-2, 5th Army Corps; Chf. G-2-C, 2d Army; Acting Asst. Chf. of Staff, G-2, 2d Army; Deputy Chf. G-2-C, G. H. Q., A. E. F. Somme Sector;

† Died Dec. 21, 1919.

Cambrai operations; St. Mihiel offensive; Toul Sector. Chevalier, Legion d'Honneur; Citation from Gen. Pershing. Three stars.

HOLBORN, LEWIS ADAMS
1st Lt., Engrs., U. S. A., A. E. F.*

HOLBROOK, WINFIELD
Entered service Oct. 2, 1918; Capt., Engrs., U. S. A., Sept. 21, 1918. Discharged Jan. 10, 1919. Camp Humphreys.

HOLLMAN, JOHN GEORGE
Entered service Aug. 15, 1917, as Capt., Engrs., N. A. Overseas service Aug., 1918–Jan., 1919. Resigned Jan. 15, 1919. Co. Comdr., 315th Engrs.; Co. Comdr., 35th Engrs.; Depot Engr. Officer, New Orleans and at Base Section No. 4, A. E. F.

HOLLYDAY, RICHARD CARMICHAEL
Entered service Mar. 14, 1894, as Lt., Jr. Grade, C. E. C., U. S. N.; Capt., C. E. C., U. S. N., Jan. 12, 1912. Public Works Officer, 5th Naval Dist. and 3d Naval Dist.

HOLMBOE, LAWRENCE SCOFIELD
2d Lt., M. T. C., U. S. A.*

HOLMES, HOWARD WHITTIER
U. S. A.*

HOLMES, THOMAS HUGHES
Entered service May 10, 1917; Capt., Engrs., N. A., Aug. 15, 1917. Overseas service, Sept. 8, 1918–July 1, 1919. Discharged July 29, 1919. Co. Comdr. and Bn. Comdr. 311th Engrs.; Dist. Road Officer, Dist. No. 1, Base Sec. No. 2, A. E. F.; C. O., U. S. troops, Limoges, France.

HOLSTLAW, CHARLES HENRY
Entered service Aug. 5, 1917, as 1st Lt., Inf., N. A. Overseas service Oct. 28, 1918–July 21, 1919. Discharged Aug. 12, 1919. With 124th Inf.; C. O., 58th Guard Co.

HOLT, ANDREW HALL
Entered service May 15, 1917; 1st Lt., E. O. R. C., June 23, 1917; Capt., E. O. R. C., Aug. 15, 1917. Overseas service Sept. 30, 1918–July 5, 1919. Discharged Aug. 14, 1919. Instr. engr. training camp; Personnel Officer, Office Director Gen. Military Railways., Office Chf. of Engrs.; Personnel Adj. 605th Engrs.; head of Civil Eng. Dept., Univ. of Beaune, France.

HONEYMAN, BRUCE RITCHIE
Entered service May 11, 1917; Capt., Engrs., U. S. A., Aug. 15, 1917. Overseas service 1918. E. O. T. C., Fort Snelling and Fort Leavenworth. Co. Comdr., 313th Engrs. Died Oct. 5, 1918, in service in France.

HOOPES, EDGAR MALIN, JR.
Entered service Sept. 9, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged June 21, 1919. Asst. to Officer in Chg. and Officer in Chg. Utilities, Camp Meade; Officer in Chg. Utilities, Camp Custis.

HOOVER, ANDREW PEARSON
Entered service June 18, 1918, as Maj. Q. M. C., Constr. Div., N. A.; Lt. Col.; Q. M. C., Constr. Div., U. S. A., May 3, 1919. Discharged Sept. 4, 1919. Associate Const. Q. M., Army Supply Base, Boston; Const. Q. M., Army Supply Base, New Orleans.

HOPKINS, WILLIAM TRENHOLM
Entered service Aug. 23, 1917; 1st Lt., F. A. R. C., Nov. 27, 1917. Overseas service Feb. 26, 1918–Aug. 24, 1919. Discharged Sept. 25, 1919. Battery Executive Officer with 76th F. A. Distinguished Service Cross. Three stars. One wound.

HOPKINSON, GEORGE MARTIN
Entered service Apr. 27, 1918, as Capt., Q. M. C., Constr. Div., N. A. Discharged Oct. 1, 1919. Asst. to Constr. Officer, Curtis Bay, Md.; Const. Officer in chg. erection shops, etc., at plant of Holt Mfg. Co., Peoria, Ill.

HORAN, HAROLD JOSEPH
Entered service Feb. 18, 1918, as Pvt., Engrs., N. A.; Cpl., Engrs., N. A., Mar. 11, 1918; Sgt., Engrs., N. A., July 4, 1918. Overseas service May 10, 1918–June 8, 1919. Discharged June 28, 1919. With 42d Engrs., later 42d Bn., 20th Engrs.

HORRIGAN, WILLIAM JAMES
Entered service Sept. 21, 1918, as 1st Lt., Q. M. C., Constr. Div., U. S. A. Asst. Utilities Officer, Camps McClellan and Knox.

HORTON, CHARLES KAAPKE
Entered service May, 1917; Capt., Engrs. N. A., Sept., 1917. Overseas service May 1918–June 1919. Discharged July 3, 1919. Asst. to Const. Q. M., Camp Bowie; Co. Comdr., 111th Engrs., 1st Corps troops, road and bridge building.

HORTON, DWIGHT FRED
Entered service Aug. 27, 1917, as Capt., Engrs., N. A.; Capt., Q. M. C., Constr. Div., N. A.; Maj., Engrs., U. S. A. Overseas service, Aug. 1, 1918–Mar. 12, 1919. Discharged Mar. 19, 1919. Const. Q. M., Camp Bowie; Co. Comdr. and Bn. Comdr., 26th Engrs.

HOUGH, DAVID LEAVITT
Entered service Apr. 17, 1918, as Maj., Q. M. C., N. A. Discharged Dec. 14, 1918. Organized 323d, 329th, 316th, 342d and 328th Labor Bns.; Army member Longshoremen's Comm., New York City.

HOWARD, CONWAY ROBINSON
Entered service June 11, 1917, as 1st Lt., E. O. R. C.; Capt., Engr. R. C., June 29, 1918. Overseas service, July 28, 1917–Sept. 8, 1919. Discharged Sept. 8, 1919. With 17th Engrs., represented Am. Relief Administration at Zagreb, Jougoslavia. Class IV, Order of St. Sava, Serbia.

HOWARD, ERNEST EMMANUEL
Entered service Oct. 4, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 23, 1918. Camp Benjamin Harrison.

HOWARD, GERALD BRANCH
1st Lt., Engrs., U. S. A., A. E. F.*

HOWARD, ROBERT CHESTER
Entered service June 11, 1918, as Capt., Engrs., N. A. Discharged Dec. 17, 1918. With 75th Engrs. in organization and preparation for overseas service, also Bn. Adj.; Bn. Adj., 138th Engrs. and 1st Bn. Overseas Training Regt.

HOWE, HERBERT FRANK
Capt., Engrs., U. S. A.*

HOWE, WILSON TYLER
Entered service May 6, 1918, as Capt., Q. M. C., Constr. Div., N. A. Asst. to Const. Q. M., Edgewood Arsenal, Army Supply Base at New Orleans, and Mexican Border Project; Acting Utilities Officer, Nogales, Ariz.; Const. Q. M. and Utilities Officer, Ft. Brown, Tex.; Const. Officer, Aberdeen Proving Grounds.

HOWELL, CLARENCE SCOTT
Capt., Engrs., U. S. A.*

HOWELL, GEORGE PIERCE
Entered service June 15, 1889; Col. (temporary), C. of E., U. S. A., Aug. 5, 1917. C. O. 210th Engrs., 10th Div., Camp Funston; Camp Mills.

HOWELL, ROBERT PARSONS †
Entered service July 28, 1918, as Capt., Q. M. C., Constr. Div., N. A. Discharged Aug. 11, 1919. Asst. to Utilities Officer, Camp Dix; Officer in Chg. Water Supply and Sewers, Camp Gordon and Port of Embarkation, Newport News.

HOWES, CYRUS PIERCE
Entered service June 21, 1918, as 1st Lt., Engrs., N. A. Overseas service, June 30, 1918–July 5, 1919. Discharged July 28, 1919. In chg. hosp. constr., Poitiers, France; in chg. detachment, Base Hosp., Paris; gen. constr., Montierchaume; with 55th Engrs.

HOYT, SIDNEY MERRILL
Entered service July 12, 1918, as Capt., Q. M. C., Constr. Div., N. A. Discharged June 30, 1920. Superv. Const. Officer in chg. constr. Arsenals and Ord. Storage Depots.

HUBER, JOSEPH EARL
Entered service May 17, 1918; Pvt., C. A. C., N. A., May 21, 1918; 2d Lt., C. A. C., U. S. A., Sept. 25, 1918. Discharged Dec. 1, 1918. With 17th Co., C. A. C.; Instr. in Gunnery, C. A. School, Ft. Monroe.

HUBER, WILLIAM THOMAS
Entered service Dec. 28, 1917, as Capt., Engrs., N. A. Discharged Feb. 1, 1919. Co. Comdr., 2d Engr. Training Regt. and 212th Engrs.

HUCKABY, MARION COLUMBUS
Entered service Aug. 25, 1917; 1st Lt., C. A. C., N. A., Nov. 27, 1917. Discharged Dec. 28, 1918. Northwest Pacific C. A. Dist. for training; with 39th Artillery.

HUDSON, HAROLD WALTON
Entered service May 2, 1917; Capt., E. O. R. C., Jan. 26, 1917; Maj. Engrs., N. A., June 2, 1918; Lt. Col., Engrs., U. S. A., Oct. 11, 1918. Overseas service July 14, 1917–Apr. 23, 1919. Discharged May 5, 1919. Co. Comdr., 11th Engrs.; Deputy Engr. Constr., Transportation Dept., A. E. F.; Engr., Constr. Staff, Director Gen. Transportation; Lt. Col. with 11th Engrs. Legion d'Honneur; Citation from Gen. Pershing for meritorious service.

HUGHES, GEORGE LEYBURN
Entered service Sept. 29, 1917, as Pvt., Inf., N. A.; Cpl., Engrs., N. A., Oct. 12, 1917; 2d Lt., Engrs., N. A., May 16, 1918. Discharged Jan. 11, 1919. With 313th Inf., 304th Engrs., Acting Adj. Engr. Replacement Troops, American Univ.; Engr. Depot, Norfolk, Va.

HUIE, IRVING VAN ARNAM
Entered service May 7, 1917; 1st Lt., E. O. R. C., June, 1917; Capt., Engrs., N. A., Jan. 1918; Maj. Engrs., U. S. A., Aug., 1918. Overseas service Aug. 7, 1917–Sept. 2, 1918. Discharged Jan. 7, 1919. With 1st Engrs. and 4th Engr. Training Regt.; Senior Instr., E. O. T. S. Regtl. citation for services in Montdidier and Marne battles.

HULBURD, LUCIUS SANFORD
Entered service Sept. 5, 1918; Capt., Engrs., U. S. A., Aug. 22, 1918. Discharged Nov. 30, 1918. Co. Comdr., 3d Engr. Training Regt., Camp Humphreys.

HULL, GEORGE BECKLEY
Entered service July 4, 1915, as Capt., Canadian Army; Maj., Canadian Forestry Troops, Sept. 10, 1916. Overseas service Aug. 20, 1915–Feb. 18, 1919. Discharged Feb. 18, 1919. 2d in command, Canadian Forestry Troops, Dist. 52. Mentioned in despatches.

† Died Sept. 29, 1920.

HULL, GORDON BURNETT GIFFORD
Entered service Sept. 10, 1915, as Lt., Royal Engrs., British Army; Capt. Royal Engrs., Sept. 15, 1917; Maj., Royal Engrs., Oct. 1, 1918; Lt. Col., Royal Engrs., June 22, 1919. Overseas service Sept. 10, 1915–Sept., 1919. Discharged Nov., 1919. Staff officer to Chf. Engr. 12th Army Corps; Water Supply Officer, 2d Corps; served in Archangel, Russia, and the Balkans. Order of the British Empire; mentioned in despatches (twice); brought to notice (once); Order of St. Stanislaus, Russia.

HULSART, CHARLES RAYMOND
Entered service May 29, 1917, as Capt., E. O. R. C.; Maj. Engrs., N. A., July 30, 1918. Overseas service July 14, 1917–Aug. 20, 1918. Discharged Feb. 15, 1919. Co. Comdr., 11th Engrs.; Sr. Instr., Engr. Training School, Camp Humphreys. Distinguished Service Cross; Military Cross, Great Britain, for Cambrai action.

HUMANN, EDWIN AUGUST
Entered service June 6, 1917, as Pvt., E. O. R. C.; Cpl., Engr. R. C., Feb. 21, 1918; M. E., Engrs., U. S. A., Nov. 27, 1918. Overseas service May 1, 1918–July 29, 1919. Discharged Aug. 6, 1919. With 4th Engrs.; topographic office, map reproduction; Vesle River, assembling bridge material; Div. Engr. Office. Four stars.

HUMPHREY, GILBERT EDWIN
Maj., Engrs., U. S. A.*

HUMPHREYS, CHARLES RAYMOND
Entered service May 8, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service May 26, 1918–May 20, 1919. Discharged June 9, 1919. Topographic Officer and Water Supply Officer, 105th Engrs.; Water Supply Officer, 2d Army Corps. Three stars.

HUMPHREYS, EWING SLOAN
Entered service Aug. 4, 1917, as 1st Lt., Engrs., N. A.; Capt., Engrs., N. A., Aug., 1918. Discharged Mar. 1, 1919. General Engr. Depot, serving in Pittsburgh Dist.; Office, Chf. of Engrs.

HUNT, CHARLES ADAMS
Entered service Apr. 8, 1918; Lt., C. E. C., U. S. N. R. F., Apr. 2, 1918. Released from active service Apr. 16, 1919. Public Works Office, 5th Naval Dist., Norfolk, Va.

HUNT, CONWAY BETHUNE
Entered service Jan 23, 1917, as Maj., E. O. R. C. With 23d Engrs., Camp Meade; member Bd. of Officers to prepare manual of instructions.

HUNT, GEORGE ALVIN
Entered service May 14, 1917; Capt., E. O. R. C., June 17, 1917; Maj. C. W. S., U. S. A., Aug., 1918; Lt. Col., C. W. S., U. S. A., Mar., 1919. Overseas service Dec. 7, 1917–Aug. 1, 1919. With 317th Engrs., Camp Sherman; Chf. Engr., Intermediate Sec., A. E. F.; Asst. Gas Officer, 3d Div.; Gas Officer, 90th Div.; Corps Gas Officer, 1st Army Corps; Army Gas Officer, 3d Army; Edgewood Arsenal; Chemical Warfare Officer, Western Dept. Five stars.

HUNT, LEIGH ANSON
Entered service Aug. 5, 1917; 1st Lt., Inf. R. C., Apr. 6, 1917; Capt., E. O. R. C., June 7, 1917; Maj., E. O. R. C., July 7, 1917. Overseas service May 2, 1918–Oct. 2, 1918. Discharged Dec. 31, 1919. With 110th Engrs. as Bn. Comdr. and Instr. Field Fortifications; Asst. Chf. of Staff, G-2, 6th Army Corps; Div. G-2, 76th Div.; Gen. Staff College, Langres, France.

HURLBUT, HINMAN BARRETT
Capt., Ord., U. S. A.*

HURLEY, JOHN PATRICK
Capt., Inf., U. S. A., A. E. F.*

HUSSON, WILLIAM MORAGNE
Entered service Nov. 28, 1916, as 2d Lt., Cavalry, U. S. A.; 1st Lt., 1917; Capt., Aug., 1917. Discharged Mar., 1918. With 17th Cavalry and 82d F. A.

HUSTED, ALVA GUY
Entered service Mar. 29, 1918, as Capt., San. C., N. A. Discharged Mar. 22, 1919. Asst. to San. Insp., Camp Wadsworth; Camp San. Engr., Camps Wadsworth and Sheridan.

HUSTON, TILLINGHAST L'HOMMEDIEU
Entered service May, 1917, as Capt., E. O. R. C.; Maj., Engr. R. C., May, 1918; Lt. Col., Engrs., U. S. A., Aug., 1918. Overseas service Aug. 1, 1917–Jan. 1, 1919. Discharged Jan., 1919. With 16th Engrs. on constr. ry. yards, Is-sur-tille; light ry. constr. with British near Arras; constr., Mesves Hosp.; constr., light ry. in Argonne; reconstr. light ry. Verdun-Sedan. Citation from Gen. Pershing for meritorious service.

HYDE, CHARLES GILMAN
Entered service June 11, 1918; Capt., San. C., U. S. A., June 8, 1918; Major., San. C., U. S. A., Oct. 23, 1918. Discharged June 12, 1919. Camp San. Engr., Camp Meade; Surgeon General's Office, Div. of Sanitation, in chg. San. Eng. Sec.

HYDE, RICHARD LEWIS
Entered service May 8, 1917; 2d Lt., E. O. R. C., Aug. 15, 1917; 1st Lt., Engrs., U. S. A., Oct. 26, 1918. Overseas service Jan. 4, 1918–July 29, 1919. Discharged Aug. 12, 1919. With 20th Engrs.; Shipping Officer with 3d Bn., 10th Engrs., in Besancon Dist.; with 72d Engrs.; Asst. to, and Office Engr., Base Engr., Base Sec. No. 1, A. E. F.

HYNDS, HAROLD DEVILLO
Entered service Sept., 1917, as 1st Lt., Aviation Sec., Sig. C., N. A.; Capt., A. S., U. S. A., Sept., 1918. Discharged Jan., 1919. Duty on Pacific Coast.

INGALLS, OWEN LOVEJOY
Entered service Apr., 1918; Maj., E. O. R. C., June, 1917. Discharged Jan., 1919. With 139th Engrs.

IRISH, LELAND WESLEY
Entered service Feb. 2, 1918, as Pvt., Engrs., N. A.; M. E., Engrs., N. A., Feb. 11, 1918. Overseas service May 10, 1918–Apr. 19, 1919. Discharged Apr. 26, 1919. With 20th and 42d Engrs.; detached service at St. Nazaire, France.

JACKSON, DUGALD CALEB
Entered service Apr. 20, 1918; Maj., E. O. R. C.; Lt. Col., Engrs., U. S. A., Feb. 13, 1919. Overseas service May 9, 1918–May 2, 1919. Discharged May 6, 1919. Ch. Engr., Technical Bd., procurement and co-ordination electric and mechanical power; Director, War Damages Bureau, Peace Comm. Chevalier, Legion d'Honneur.

JACKSON, JESSE AARON
Entered service July 23, 1917; Maj., Sig. R. C., Apr. 5, 1917. Overseas service Nov. 26, 1917–Feb. 13, 1919. Discharged Apr. 3, 1919. 1st Corps School, Langres, France; special duty, 1st Div., C. O., 116th Field Sig. Bn.

JACKSON, WILLIAM BENJAMIN
Entered service Apr. 19, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged Dec. 31, 1918. Utilities Officer, Camp Merritt; in chg. Const. Q. M. Dept., Port of Embarkation, New York.

JACOBOSKY, GILBERT GARFIELD
Entered service July 15, 1917, as Capt., F. A., N. A.; Capt., Engrs., U. S. A.; Maj., Engrs., U. S. A., Apr. 8, 1919. Overseas service June 29, 1918–July 6, 1919. Discharged Aug. 11, 1919. With 109th F. A. as Co. Comdr.; 108th F. A. as Bn. and Regtl. Adj.; with 55th Engrs as Bn. Comdr. in chg. road work; Instr. in bridges and foundations, Univ. of Beaune, France.

JACOBS, JOSEPH
Entered service Oct. 22, 1917, as Maj., Engrs., N. A. Overseas service Feb. 16, 1918–Sept. 16, 1919. Discharged Oct. 10, 1919. C. O., 509th Engrs.; special duty as co-ordinating and personnel officer to Sec. Engr., Base Sec. No. 1, A. E. F.; special duty in road and quarry work, Intermediate Sec., Paris Dist., A. E. F.; Executive Officer, Pershing Stadium Project, Paris.

JACOBSEN, HANS PETTER RUDE
Entered service Sept. 7, 1918; Capt., Engrs., N. A., July 15, 1918. Discharged Jan. 9, 1919. Co. Comdr., 124th Engrs.

JACOBSON, ALFRED LEON
Entered service Oct. 1, 1902; Capt., F. A., French Army, July 15, 1915. C. O., 12th Btry., 45th F. A. and 3d Group, 15th F. A.; attached to American Artillery School, Saumur, France; attached to 42d (American) Div. Chevalier, Legion d'Honneur; cited in corps and army orders and in orders of French Mission with 1st American Army; Distinguished Service Cross. Wounded Sept. 25, 1916, Somme; Nov. 6, 1916, Somme; May 11, 1917. Aisne. Five stars.

JACQUES, HENRY LOUIS
Entered service Dec. 28, 1917; Capt., Engrs., N. A., May 1, 1918. Overseas service Aug. 31, 1918–Mar. 20, 1919. Discharged Apr. 26, 1919. Regtl. Personnel Officer, Co. Comdr. and Supply Officer, 27th Engrs.

JAENICKE, WILLIAM HUGO
Entered service Mar. 1, 1918, as Pvt., Engrs., N. A.; 2d Lt., Engrs., N. A., June 16, 1918; 1st Lt., Engrs., U. S. A., Oct. 23, 1918. Discharged Feb. 25, 1919. Instr., E. O. T. S., Camp Humphreys.

JAMES, ALFRED RANDOLPH
Entered service June 8, 1918, as 1st Lt., Q. M. C., N. A.; Capt., Q. M. C., U. S. A., June 2, 1919. Discharged Sept. 16, 1919. Installation of standard accounting systems in supply bases throughout U. S.; Chf. of Accounting Methods and Service Branch, Office of the Dir. of Storage, Q. M. Gen. Office.

JAMES, FRANK TROWBRIDGE
Entered service July 30, 1918; Capt., E. O. R. C., Oct. 4, 1917. Discharged Dec. 5, 1918. E. O. T. S., Camp Humphreys.

JAMES, ROBERT LANE
Capt., Engrs., U. S. A., A. E. F.*

JAQUES, JACOB DUNCAN
Entered service Apr. 6, 1918, as Lt., C. E. C., U. S. N. R. F. Released from active service, Mar. 31, 1919. Asst. to Dist. Public Works Officer, 1st Naval Dist.

JARVIS, CLARENCE SYLVESTER
Entered service Oct. 26, 1918, as Capt., Engrs., U. S. A. Div. of Physical Reconstr., Surgeon General's Office, Denver, Colo.; Constr. Div. on Frankford Arsenal, Pa., Benicia Arsenal and Ross Field, Cal.

JASPER, THOMAS McLEAN
Entered service May, 1915, as 2d Lt., Royal F. A., British Army; Lt., Royal F. A., 1917; Capt., Royal F. A., 1917. Discharged May, 1919. With 172d Royal F. A.; Officers' Train-

ing School, England; with Ministry of Munitions, London; At Arras, Somme, with 36th Div. (Ulster) Artillery, 2d and 3d British Armies. Wounded twice.

JENKINS, CHARLES MELVILLE
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., June 19, 1917; Capt., Engrs., U. S. A., July 30, 1918. Overseas service Jan. 3, 1918–Sept. 8, 1918. Discharged Nov. 30, 1918. With 20th Engrs. as Bn. Supply Officer, and Dist. Supply Officer at Dijon, France; Instr. in organizing Sapper Regts.; Office, Chf. of Engrs., in organization Ry. Personnel Dept.

JENKINS, JENKS BUFFUM
Entered service Dec. 28, 1917; Maj., E. O. R. C., Feb. 16, 1917. Discharged Aug. 16, 1919. Asst. to Officer in Chg. cantonment constr., Washington, D. C.; Const. Q. M., U. S. General Hosps. No. 19, 12, 18, and 23; Asst. to Chf. of Constr. Div., Washington, D. C.

JENNINGS, CHARLES AUGUSTUS
Entered service Sept. 21, 1918; Capt., Q. M. C., Constr. Div., U. S. A., Sept. 12, 1918. Discharged Mar. 22, 1919. Officer in chg. Water and Sewers, Ft. Oglethorpe; special work at Camp MacArthur, in connection with contaminated water supply.

JENNINGS, PERCY JOHN
Entered service March 17, 1916, as Lt., Canadian Pioneers; Capt., Canadian Pioneers, Apr. 1, 1916; transferred to Royal Engrs., July 28, 1917; Maj., Royal Engrs., Mar. 13, 1918. Overseas service Sept. 12, 1916–July 12, 1919. Discharged July 12, 1919. With 4th Canadian Pioneers; Deputy Asst. Director, Inland Water Transport, East African Expeditionary Force. Order of the British Empire.

JERRARD, LEIGH PATTERSON
Entered service June 20, 1917, as Regtl. Supply Sgt., F. A.; 2d Lt., F. A., N. A., Aug. 20, 1917; 1st Lt., F. A., N. A., May 25, 1918; Capt., F. A., U. S. A., Oct. 25, 1918. Overseas service Oct. 18, 1917–Apr. 26, 1919. Discharged May 29, 1919. With 149th F. A.; Intelligence Officer, 67th F. A. Brig. Four stars.

JERVEY, HENRY
Entered service Aug. 16, 1916; Col., C. of E., U. S. A., May 15, 1917; Brig. Gen., N. A., Aug. 5, 1917; Maj. Gen., U. S. A., Oct. 1, 1918. Reverted to permanent rank of Brig. Gen., U. S. A., May 1, 1920. Asst. Instr. Army War College; on duty American Univ. Training Camp; Commandant, Engr. School, Washington Barracks; Col., 66th F. A. Brig., Camp Fremont and Camp Greene; Acting C. O., 41st Div.; C. G. 66th F. A. Brig.; Acting Director of Operations, and Director of Operations, Gen. Staff; Recorder, Bd. of Officers considering persons entitled to Medals of Honor. Distinguished Service Medal; Commandeur, Legion d'Honneur; Grand Officer, Order of Leopold, Belgium; Companion of the Bath, Great Britain.

JESSUP, WALTER EDGAR
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., June 18, 1917. Overseas service June 17, 1918–Aug. 15, 1919. Discharged Sept. 3, 1919. Instr., Engr. Training, Camp Lee; with 39th Engrs. in chg. locomotive roundhouse at Nevers, France; Personnel Adj., 15th Grand Div., T. C.

JEWELL, ALBERT HARTWELL
Entered service Oct. 29, 1917; 1st Lt., San. C., N. A., Sept. 5, 1917. Overseas service Aug. 17, 1918–Mar. 23, 1919. Discharged Apr. 18, 1919. San. Officer, Selfridge Aviation Field; with 26th Engrs. at Camp Dix and overseas; water analysis laboratory, Paris; with 107th Engrs. in reconnaissance of water supplies, Luxembourg and Germany; Meuse-Argonne offensive. One star.

JEWETT, THOMAS EDWARD
Entered service Apr. 21, 1918, as Capt., Q. M. C., Constr. Div., N. A. Discharged Jan. 17, 1919. Asst. Supervising Const. Q. M., terminals and warehouses.

JOHNSON, FRANCIS WHITTIER
Entered service June 26, 1918, as Pvt., C. A. C. training camp; 2d Lt., C. A. C., U. S. A., Sept. 25, 1918. Discharged Dec. 10, 1918. Orientation and Reconnaissance Officer, 37th Artillery.

JOHNSON, FRANK MELVIN S.
Entered service June 5, 1917, as 2d Lt., C. of E., U. S. A.; 1st Lt., C. of E., U. S. A., Sept. 17, 1917; Capt., C. of E., U. S. A., Oct. 10, 1917; Maj., Engrs., U. S. A., Sept. 24, 1918. Overseas service Apr. 30, 1918–Sept. 12, 1918. With 4th Engrs.; Asst. Ch. of Staff, 16th Div. Permanent rank, Capt., C. of E., U. S. A. Three stars.

JOHNSON, GEORGE ARTHUR
Entered service June 17, 1918, as Maj., Q. M. C., Constr. Div., N. A.; Lt. Col., Q. M. C., Constr. Div., U. S. A., Nov. 6, 1918; Col., Q. M. C., Constr. Div., U. S. A., July 31, 1919. Discharged Mar. 12, 1920. Executive Officer, Utilities Div., Constr. Div., Washington, D. C.

JOHNSON, HALBERT THEODORE
Sgt., Engrs., U. S. A.*

JOHNSON, HARVEY STONE
Entered service May, 1917; 2d Lt., F. A., N. A., Aug., 1917; 1st Lt., F. A., N. A., Jan., 1918. Overseas service May, 1918–May, 1919. Discharged June, 1919. With 309th F. A.; Aerial Observation School; Div. Observer.

JOHNSON, HOLLISTER
Entered service Sept. 2, 1917; 2d Lt., E. O. R. C., July 26, 1917; Capt., C. of E., U. S. A., July 7, 1918. Overseas service Nov. 12, 1917–June 29, 1919. Resigned Sept. 25, 1919. With 20th Engrs.; with H. Q. Div. Constr. and Forestry, S. O. S., A. E. F.; H. Q., 1st Army; with 115th Engrs. One star.

JOHNSON, JOHN MONROE
Entered service June 1, 1917, as Maj., Engrs., N. G.; Lt. Col., C. of E., U. S. A., Apr., 1918; Col., C. of E., U. S. A., Oct., 1918. Overseas service Oct 18, 1917–May 1, 1919. Discharged June 4, 1919. C. O., 117th Engrs.; Chf. Engr., 42d Div. Four stars.

JOHNSON, LOUIS RAUB
Entered service Sept. 2, 1917, as Capt., Engrs., N. A.. Overseas service July 9, 1918–July 16, 1919. Discharged Aug. 4, 1919. Co. Comdr. and Bn. Comdr., 516 Engrs., Engr. Depot, Gievres, France.

JOHNSTON, THOMAS STEWART
Entered service Nov. 4, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 4, 1918. E. O. T. S., Camp Humphreys.

JOHNSTONE, LESLIE INGALLS
Entered service Sept. 19, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 24, 1918. E. O. T. S., Camp Humphreys.

JONAH, FRANK GILBERT
Entered service June 7, 1917; Maj., E. O. R. C., Jan. 23, 1917; Lt. Col., Engrs., U. S. A., Oct. 3, 1918. Overseas service July 28, 1917–Jan. 13, 1919. Discharged Jan. 18, 1919 With 12th Engrs.; with Div. Light Ry. as Chf. Engr., G. H. Q., A. E. F. Citation; personal letter of commendation from Gen. Pershing.

JONES, GRANDVILLE REYNARD
Entered service Nov. 19, 1918; Capt., San. C., U. S. A., Nov. 11, 1918. M. O. T. C., Camp Greenleaf; Camp San. Engr., Camp Benning. Died Dec. 22, 1918, in service at Camp Benning, Ga.

JONES, HARRY EDWARD
Entered service Apr. 10, 1918, as Pvt., Sig. C., N. A. Overseas service Sept. 14, 1918–Jan. 1, 1919. Discharged Jan. 15, 1919. With Meteorological Sec., Sig. C.; Camp de Meucon, Vannes, France.

JONES, HARVEY WILLARD
Entered service Aug. 12, 1918; Capt., Q. M. C., Constr. Div., N. A., Aug. 2, 1918. Discharged May 12, 1919. Utilities Officer, Camp Lee.

JONES, HENRY LLEWELLYN
Entered service May 4, 1917, as Maj., E. O. R. C. Discharged Dec. 15, 1917.

JONES, JOHN HENRY
Capt., Canadian Pioneers, B. E. F.*

JONES, JONATHAN
Entered service July 30, 1917, as Capt., Engrs., N. A.; Maj., Engrs. U. S. A., Oct. 19, 1918. Overseas service Mar. 30, 1918–July 9, 1919. Discharged July 29, 1919. Co. Comdr. and Comdr., Eng. Motor Train, 23d Engrs.; 1st Army, Meuse-Argonne offensive. One star.

JONES, LEWIS ALLEN
Entered service Aug. 31, 1917; Capt., E. O. R. C., June 12, 1917. Overseas service Apr. 29, 1918–Apr. 1, 1919. Discharged Apr. 2, 1919. Co. Comdr., 17th and 514th Engrs.; Resident Engr., Hosp. constr., London.

JONES, PAUL SIDNEY
Entered service May 15, 1918, as Pvt., F. A., N. A.; Pvt., Engrs., N. A., Aug., 1918; 2d Lt., Engrs., U. S. A., Oct., 1918. Discharged Feb. 21, 1919. With 5th Engr. Training Regt.; trained newly drafted engr. troops.

JONES, PERCIVAL CHARLES
Entered service Nov. 18, 1917, as Pvt., Engrs., N. A.; Cpl., Engrs., N. A., Feb. 18, 1918; Sgt., Engrs., U. S. A., Apr. 15, 1919; 2d Lt., Engrs. U. S. A., May 22, 1919. Overseas service Mar. 27, 1918–June 9, 1919. Discharged June 20, 1919. With 23d Engrs.; road work in St. Mihiel and Meuse-Argonne offensives. Two stars.

JONES, PERCY FRANCIS
Capt., Engrs., U. S. A.*

JONES, SIDNEY GARDNER
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service Feb. 18, 1918–May 1, 1919. Discharged May 25, 1919. Co. Comdr., 35th Engrs. and various T. C. Cos. included in 35th Engrs.; Salvage Officer and San. Officer, Camp Aytre, La Rochelle, France.

JORDAN, HARRY EDWARD
Entered service July 26, 1918, as Capt., Q. M. C., Constr. Div., N. A.; Maj., Q. M. C., Constr. Div., U. S. A., June 2, 1919. Discharged Sept. 9, 1919. Southeastern Dist., Insp. of Utilities Service; Utilities Officer, Const. Q. M., and Sr. Officer of Real Estate Damage Claims Bd., Camp Sevier, S. C.

JOSEPH, JACOB
Entered service Apr. 20, 1918, as Pvt., Sig. C., N. A. Overseas service Aug. 5, 1918–Apr. 19, 1919. Discharged Apr. 29, 1919. With Meteorological Sec., Sig. C.

JOUINE, GEORGES PIERRE FERDINAND
Entered service Sept. 9, 1914, as Pvt., F. A., French Army; Cpl., F. A., Sept., 1915; Sgt., F. A., Mar., 1916; Sous-Lt. Tanks Corps, May, 1917. With French Army from enlistment to Mar. 14, 1919. With Trench Mortar Battery, 58th F. A.; with 81st Heavy F. A., Tank Corps Dept. Croix de Guerre, with three silver stars. Both units with which he served twice cited and awarded Croix de Guerre with Palm, allowing members to wear fourragère red and green. Wounded five times.

JUDSON, WILLIAM VOORHEES
Entered service as Cadet, U. S. M. A., June 15, 1884; Brig. Gen., N. A., Aug. 5, 1917. Overseas service, May 15, 1917–Feb. 22, 1918. Returned to rank of Col., C. of E., U. S. A., Feb. 6, 1919. Chf. of Military Mission to Russia. Order of St. Anna and Order of St. Stanislaus, Russia.

KACKLEY, WALTER JOHN
Entered service May 1, 1918, as Lt., Jr. Grade, C. E. C., U. S. N. R. F.; Lt., C. E. C., U. S. N. R. F., Dec. 1, 1918. Released from active service May 1, 1919. Project Officer and Asst. Technical Officer on design, Public Works Dept., Philadelphia Navy Yard.

KANE, DANIEL COUGHLIN
1st Lt., Engrs., U. S. A.*

KANE, IRVING PATTERSON
Maj., Engrs., U. S. A., A. E. F.*

KARNOPP, EDWIN BENJAMIN
Entered service Apr., 1918; Capt., Engrs., N. A., June, 1918. Overseas service July, 1918–Mar., 1919. Discharged Sept., 1919. With 22d Engrs., on light ry. constr. in Meuse-Argonne Sector.

KASTENHUBER, EDWIN GUSTAV, JR.
Entered service Nov. 28, 1917, as 1st Lt., San. C., N. A.; Capt., San. C. N. A., March 4, 1918. Overseas service July 8, 1918–July 8, 1919. Discharged July 30, 1919. San. Engr. to Div. Surgeon, 36th Div., Camp Bowie; C. O., San. Squadron 55, A. S., S. O. S., A. E. F.

KAUFMANN, ERNST GUSTAV
Entered service Feb. 27, 1918, as Pvt., A. S., N. A.; Sgt., 1st Class, A. S., U. S. A., Nov. 8, 1918; 2d Lt., Q. M. C., U. S. A., Oct. 22, 1918. Overseas service Oct. 21, 1918–July 20, 1919. Discharged Aug. 6, 1919. With 499th Aero Squadron, 307th Supply Train, Q. M. C.

KEENAN, JOHN THOMAS
Entered service Aug. 4, 1917, as Maj., Engrs., N. A. Overseas service July 1, 1918–July 12, 1919. Discharged July 30, 1919. Bn. Comdr., 28th Engrs., constr. light rys. and roads, 1st Army, St. Mihiel, Argonne, Langres, Aisne, France.

KEITH, GERALD MARCY
Entered service May 8, 1917; 2d Lt., E. O. R. C., June 28, 1917; 1st Lt., Engrs., U. S. A., June 3, 1918. Overseas service Dec. 11, 1917–May 6, 1918. Discharged Nov. 1, 1919. With 301st Engrs., Camp Devens; with 116th Engrs. in France; student, 2d Corps School; with 2d Engrs. in Ansauville Sector doing sapper duty; Instr., E. O. T. S., Camps Lee and Humphreys; constr., Camp Humphreys.

KELLER, ARTHUR RIPONT
Entered service June, 1918, as Capt., E. O. R. C. Discharged July 23, 1919. Training camp at Camps Lee and Humphreys; duty under Director Gen., Military Rys., Washington, D. C.

KELLER, CHARLES
Entered service June 11, 1890; Lt. Col., C. of E., U. S. A., at declaration of war; Col., Engrs., N. A., Aug. 5, 1917; Brig. Gen., N. A., Feb. 26, 1918. Overseas service Sept. 24, 1918–June 19, 1919. Returned to rank of Col., C. of E., U. S. A., June 30, 1919. Asst. to Chf. of Engrs., U. S. A., Power Sec., War Industries Bd.; Secy. and Executive, Comm. on Inland Waterways, R. R. Administration; Deputy Ch. Engr., A. E. F. Distinguished Service Medal; Officier, Legion d'Honneur.

KELLEY, JAMES AUGUSTUS
Entered service June 10, 1918; Maj., Engrs., Constr. Div., U. S. A., Aug. 17, 1918. Discharged Jan. 20, 1919. In chg. of surveys and constr., warehouses, docks and rys., Camps Dix and Merritt, and Norfolk, Va.

KELLY, EARL WALLACE
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 13, 1917. Overseas service June 22, 1918–July 14, 1919. Discharged Aug. 4, 1919. Co. Comdr., 302d Bn. Tank Corps; Co. Comdr. Camp Pike Inf. Replacements; Co. Comdr., 116th Engrs.; Co. Comdr. 1st Army Punishment Bn.; with 1st Engrs.; Executive Officer, Coblenz Leave Area. One star.

KELLY, HUGH AMBROSE
Entered service Sept. 5, 1917, as Pvt., Inf., N. A.; 1st Sgt., Inf., N. A.; 2d Lt., Inf., N. A., June 1, 1918; 2d Lt., Engrs., U. S. A., July 26, 1918. Discharged Feb. 5, 1919. With 312th Inf.; O. T. S., Camp Dix; Co. Comdr., 153d Depot Brig.; E. O. T. S., 1st Engr. Replacement Regt., Washington Barracks; with 214th Engrs.

KELLY, WILLIAM
Entered service June, 1895; Col., Engrs., N. A., Aug. 5, 1917. Overseas service Oct. 18, 1917–Jan. 24, 1920. Returned to permanent grade of Lt. Col., C. of E. U. S. A. C. O., 117th Engrs.; Chf. Engr., 42d Div.; Chf. Engr., 4th Army Corps; C. O., Base Secs. No. 7 and 9; Deputy Chf. of Staff, American Forces in France; duty with Peace Commission. Distinguished Service Medal; Officier, Legion d'Honneur. Four stars.

KEMP, JOHN EDWARD
Entered service Sept. 1, 1917, as Capt., E. O. R. C. Discharged Nov. 30, 1917, to enter civilian service overseas.

KEYS, EDWARD ALLEN
Entered service Aug. 8, 1917, as Capt., Engrs., N. A.; Maj., Engrs., U. S. A., Apr. 9, 1919. Overseas service Mar. 20, 1918–June 10, 1919. With 511th Engrs. on R. R. work at Nevers, storage depot constr. at Montierchaume, and road work, Tours Dist., France.

KHACHADOORIAN, HAROOTUN HOVHANNES
1st Lt., Engrs., U. S. A., A. E. F.*

KIMBALL, HERBERT SAWYER
Entered service June 26, 1917, as Capt., Ord., U. S. A. Discharged Feb. 8, 1919. Head of Engr. Sec., Nitrate Div., Ord. Dept., U. S. A.; Office, Chf. of Ord.

KING, CLIFFORD MARSHALL
Entered service Sept. 25, 1917; Capt., Engr. R. C., Aug. 15, 1917. Overseas service July 9, 1918–June 18, 1919. Discharged July 14, 1919. Asst. Div. Engr., 88th Div.; Co. Comdr., and C. O., 528 Engrs.; Resident Engr. constr. and maintenance light rys., Toul Sector; St. Mihiel and Meuse-Argonne offensives. Two stars.

KING, ERIC TURE
Entered service July 6, 1917; Capt., Eng. R. C., May 16, 1917; Maj., Q. M. C. Constr. Div., N. A., Mar. 22, 1918. Discharged Sept. 21, 1919. In chg. water supply constr., Camp Upton; Asst. to Officer in Chg. Constr. Div., Washington, D. C., in chg. procurement of supplies.

KING, HOWARD LANGDON
Entered service May 12, 1917; 2d Lt., Engrs., N. A., Aug. 15, 1917; 1st Lt., Engrs., U. S. A., Oct. 8, 1918. Overseas service June 28, 1918–Mar. 20, 1919. Discharged Apr. 4, 1919. With 27th Engrs. in Marne-Aisne, St. Mihiel and Meuse-Argonne offensives. Three stars.

KING, THOMAS RICHARD
Entered service Nov. 12, 1917; 1st Lt., Engrs., N. A., Aug. 4, 1917. Overseas service Feb. 16, 1918–Apr. 16, 1919. Discharged May 24, 1919. With 116th and 316th Engrs.

KING, WINFIELD SCOTT
Entered service June 12, 1917, as Capt., Q. M. C. Const. Q. M., Ft. Benjamin Harrison

KINGSLEY, EDGAR ALBERT
Entered service May, 1917; Maj., E. O. R. C., July 10, 1917. Overseas service Sept. 26, 1917–July 13, 1919. Discharged Aug. 4, 1919. Assisted in organization of Road Dept., S. O. S., A. E. F.; Supt of Roads, Intermediate Sec., A. E. F. Citation and meritorious service medal, France.

KINGSLEY, GEORGE
Entered service June 14, 1918, as Capt., E. O. R. C. Discharged Sept. 19, 1919. Designing, purchasing and shipping cargo handling devices for debarkation ports in France, and contract settlements in Office of Chf. of Engrs.

KINNE, GEORGE WHITNEY
Entered service Sept. 2, 1917; Capt., Engrs., N. A., Aug. 8, 1917. Overseas service Mar. 14, 1918–Sept. 15, 1919. Discharged Oct. 4, 1919. Office, Director Gen., Military Rys.; with Sec. Engr., Base Sec. No. 2; Depot Engr. Officer, Montierchaume.

KINNEAR, LAWRENCE WILSON
Entered service Jan. 19, 1918, as 2d Lt., Sig. R. C.; 1st Lt., A. S. A., U. S. A., Oct. 1, 1918. Discharged Feb. 15, 1919. With Officer in Chg. Constr., A. S.

KIPP, FREDERICK MARTIN, JR.
Entered service Sept. 2, 1917; 2d Lt., E. O. R. C., July 5, 1917; 1st Lt., Engrs., U. S. A., May 7, 1920. Overseas service May 10, 1918–June 2, 1920. Discharged June 27, 1920. With 33d Engrs. in constr. work, Intermediate Sec., S. O. S., A. E. F.

KIRBY, LUTHER HILL
Entered service July 18, 1917; Capt., E. O. R. C., June 23, 1917. Discharged Dec. 30, 1918. Assigned to Constr. Div. in constr. of various camps and base hosps. in U. S.

KIRSCHNER, CHARLES
Entered service May 20, 1918, as Pvt., Engrs., N. A. Discharged Dec. 11, 1918. With 472d Engrs. and E. O. T. S.

KISSACK, ALFRED BROUGHTON
Entered service Sept. 11, 1917; Capt., Engrs., N. A., July 30, 1917; Maj., Engrs., U. S. A., Sept. 21, 1918. Overseas service May 8, 1918–July 12, 1919. With 515th Engrs. in chg. constr. at Gievres and Montierchaume, France; Superv. Engr. of Roads, two French depts.

KITCHEN, ERNEST
Entered service Jan. 5, 1918; 1st Lt., Engrs., N. A., Aug. 11, 1917; Capt., Q. M. C., Constr. Div., U. S. A., Aug. 24, 1918. Engr. Officer, Harwood Mills, Va., Water Development; Const. Q. M., Coast Defenses, Northern New England, and Army and Navy Gen. Hosp., Hot Springs, Ark.

KITTREDGE, FRANK ALVAH
Entered service Mar. 1, 1918, as 1st Lt., Engrs., N. A.; Capt., Engrs., N. A., May 8, 1918. Overseas service, May 21, 1918–June 24, 1919. Discharged July 19, 1919. Co. Comdr., 43d Engrs., road and bridge work.

KITTREDGE, HARRY CHANDLER
Entered service Aug., 1918, as Capt., Engrs., U. S. A. Discharged Dec. 10, 1918. E. O. T. S., Camp Humphreys.

KITTS, JOSEPH ARTHUR
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 2, 1917. Overseas service Jan. 10, 1918–Feb. 9, 1919. Discharged Feb. 12, 1919. Instr. of Rigging, Camp Kearny; Post Engr. Officer, Blois, France; Executive Officer, Perigueux Hosp. constr.; Constr. Supply Officer, Is-sur-Tille.

KLAPP, EUGENE
Entered service Sept., 1918, as Maj., Engrs., U. S. A. Discharged Dec. 18, 1918. Camp Shelby.

KLEINSCHMIDT, HENRY SCHWING
Entered service Sept. 20, 1918, as Capt., Engrs., U. S. A. Discharged Jan. 11, 1919. E. O. T. S., Camp Humphreys.

KLINCK, JOHN HENRY
Maj., Q. M. C., U. S. A.*

KLINGNER, LOUIS WILLIAM
Entered service Nov., 1914; Lt., Canadian Engrs., Jan. 21, 1915; Capt., Canadian Engrs., May 24, 1918. Overseas service Mar. 11, 1916–July 4, 1919. Released from active service, July 6, 1919. With 2d Field Co., Canadian Engrs.; with 10th Bn., 4th Canadian Div.; Staff Capt., Canadian Engrs.; Instr., Toronto; in chg. constr. Niagara Camp; C. O., Depot Co. Canadian Eng. Training Depot, Ottawa. With B. E. F. at Ypres salient, Somme, Vimy Ridge, La Coulotte, Lens, Hill 70, Passchendaele, Arras, Amiens, Drocourt Queant, Canal du Nord, Cambrai, Denain, Valenciennes, Mons. Military Cross, Great Britain.

KLUEGEL, HARRY ALLARDT
Entered service Jan. 28, 1918; Capt., Q. M. C., N. A., Jan. 10, 1918; Maj., Q. M. C., Constr. Div., U. S. A., Nov. 1, 1918. Asst. Const. Q. M. and Utilities Officer, Camp Lewis; Const. Q. M., and Utilities Officer, Camp Fremont and Walter Reed Hosp.; Asst. Dept. Utilities Officer, H. Q., Western Dept., San Francisco, Cal.

KNAP, EDGAR DAY
Entered service Nov. 28, 1917, as Capt., Engrs., N. A. Resigned Mar. 27, 1918. Co. Comdr., 23d Engrs. (Served overseas with Y. M. C. A.)

KNIGHT, GERALD WILSON
Entered service May 8, 1917; Capt., Engrs., N. A., July 11, 1917. Overseas service Oct. 18, 1917–Feb. 22, 1919. Discharged May 5, 1919. Office, Chf. of Engrs., requisitioning water supply materials for overseas; Co. Comdr., 26th Engrs.; attached to G. H. Q., A. E. F., in Water Supply Div.; Adj. to Chf. Engr., 2d Army. Citation from Gen. Bullard for meritorious service.

KNISKERN, PHILIP WHEELER
Entered service Sept. 2, 1917; Capt., Engrs., N. A., July 10, 1917. Released from active service Sept. 6, 1919. Assigned to duty with Constr. Div.; Const. Q. M. at Chemical Plant No. 4, Saltville, Va.; Const. Q. M., Kingsport Plant, Edgewood Arsenal, Kingsport, Tenn.

KNOLLMAN, ENNO PAUL
Entered service Dec. 15, 1917, as Seaman, 2d Class, U. S. N. R. F.; Ensign, U. S. N. R. F., May, 1918; Lt., Jr. Grade, U. S. N. R. F., Mar., 1919. Released from active service Oct. 20, 1919. Ord. design, U. S. Naval Gun Factory, Washington, D. C.

KNOST, WILLIAM ARNOLD
Entered service Aug. 26, 1917; 1st Lt., C. A. C., N. A., Nov. 27, 1917. Overseas service June 10, 1918–Aug. 26, 1919. Discharged Sept. 15, 1919. Coast defense, Los Angeles, Cal.; Co. Comdr., 53d Ammunition Train; Co. Comdr., 55th Ammunition Train; duty at embarkation camp, St. Nazaire, France.

KNOX, JEAN HOWARD
Entered service Apr. 14, 1918; Lt., C. E. C., U. S. N. R. F., Apr. 5, 1918; Lt. Comdr., C. E. C., U. S. N. R. F., June 6, 1919. Overseas service Apr. 24, 1918–Feb. 11, 1919. Released from active service, Nov. 1, 1919. Public Works Officer, Naval Base, Pauillac and Paimboeuf, France; Aide to Staff of Aviation, Brest, France; Public Works Officer, Naval Air Sta., Brunswick, Ga.; Asst. Public Works Officer, Naval Base, Norfolk, Va.

KOHL, FRANK EDWARD, JR.
Entered service Dec. 9, 1917, as Pvt., Engrs., N. A.; Cpl., Engrs., N. A., Mar. 1, 1918; Sgt., Engrs., N. A., July 1, 1918. Overseas service Mar. 28, 1918–May 2, 1919. Discharged May 10, 1919. With 302d Engrs.; E. O. T. S., Langres, France, qualified for 2d Lt.

KOOP, LOUIS DIETRICH
Entered service June 24, 1917, as Lt., E. O. R. C.; Capt., Engr. R. C., May, 1918; Maj., Engrs., U. S. A., Feb., 1919. Discharged Feb., 1919. With 102d Engrs. at Camp Wadsworth; Const. Q. M., Camp Wadsworth; with 211th Engrs., Camp Meade.

KOSS, GEORGE WALTER
Capt., Engrs., U. S. A.*

KRACH, FRED ROY
Entered service Oct. 27, 1917, as Pvt., Engrs., N. A.; Pvt., 1st Class, Engrs., N. A., Oct., 1918. Overseas service Jan. 23, 1918–June 8, 1919. Discharged June 20, 1919. With 23d Engrs. Three stars.

KRIGBAUM, LOWELL GAYNOR
Entered service Sept. 2, 1917, as 1st Lt., Engrs., N. A.; Capt., Engrs., U. S. A., Oct. 21, 1918. Overseas service Jan. 7, 1918–June 27, 1919. Discharged July 2, 1919. With 109th Engrs.; on duty in Div. Constr. and Forestry, S. O. S., A. E. F.

KROMER, CLARENCE HERBERT
Entered service Oct. 27, 1918, as 1st Lt., Engrs., U. S. A. Discharged Feb. 15, 1919. With 403d Engrs.; E. O. T. S., Camp Humphreys.

KUTZ, CHARLES WILLAUER
Entered service in 1889; Lt. Col., C. of E., U. S. A., at declaration of war; Col., Engrs., N. A., Sept., 1917; Brig. Gen., N. A., June 26, 1918. Returned to permanent grade of Col., C. of E., U. S. A. Overseas service July 23, 1917–July 31, 1918. Asst. to Chf. Engr., A. E. F.; C. O., 13th Engrs.; Asst. Chf. of Staff, S. O. S., A. E. F.; C. O., Camp Humphreys. Croix de Guerre, with Palm; Officier, Legion d'Honneur.

KYLE, RALPH BRIGGS
Entered service July 27, 1917; 1st Lt., Engrs., N. A., Sept. 2, 1917. Discharged Mar. 15, 1918. Instr. in Reconnaissance, E. O. T. S., Ft. Leavenworth; Inf. Instr., trench and dugout constr., Camp Funston.

LaBACH, PAUL MAYER
Entered service Feb. 15, 1918; Maj., Engrs., N. A., Jan. 31, 1918. Discharged July 28, 1919. Overseas service Feb. 26, 1918–June 27, 1919. Engr., Water Supply, T. C., in chg. territory from base ports to the Rhine. Diploma from Gen. Pershing for meritorious service. One star.

LABSAP, ALFRED HARRY
Entered service May, 1917; 1st Lt., 114th Engrs., N. A., Aug. 17, 1917. Overseas service July 29, 1918–May 5, 1919. Discharged June 5, 1919. With 114th Engrs., road building, rifle range constr. and training of engr. troops at Camp Beauregard. With 1st Army Corps, Meuse-Argonne offensive.

LAKE, EDWARD NELSON
Entered service June 8, 1918, as Maj., Q. M. C., Constr. Div. Discharged Apr. 4, 1919. In chg. constr. Proving Grounds, Scituate, Mass., and Toluol Recovery Plant, Everett, Mass.

LAMB, LYMAN CALVIN
Capt., Engrs., U. S. A.*

LAMB, WILLIAM ALFRED
Entered service Oct. 30, 1918, as Pvt., C. A. C., U. S. A. Discharged Feb. 6, 1919. C. A. O. T. S., Fort Monroe.

LAMBERT, BYRON JAMES
Entered service Nov. 17, 1917. Maj., Engrs., N. A., Dec. 1917. Overseas service Mar. 30, 1918–Dec. 22, 1918. Discharged Jan. 6, 1919. C. O., 3d Bn. 23d Engrs.; Bridge Engr., 1st Army; Asst. to Chf. of Offense Div., C. W. S. Two stars.

LAMPHERE, FRANK ELMER
Entered service June 13, 1917, as Maj., Q. M. C., Cantonment Div.; Col., Q. M. C., Constr. Div., Mar. 18, 1918. Discharged June 30, 1920. Const. Q. M., at Camp Taylor, Port Newark Terminal and Charleston, S. C., Port Terminal, including extensive incidental constr.; in chg. Eng. Sec., Constr. Div.

LAMSON, WILLIAM MATHER
Entered service May 8, 1917; Capt., E. O. R. C., Jan. 23, 1917; Maj., E. O. R. C., Aug. 15, 1917. Overseas service July 29, 1918–July 22, 1919. Discharged Aug. 13, 1919. C. O., 1st Bn. 306th Engrs., U. S. and France; Student Officer, Army Gen. Staff College, France; Supt. Bldgs. and Grounds, H. Q., A. E. F. Univ., Beaune, France. One star.

LANCASHIRE, FOREST HENRY
Entered service May 16, 1918, as Capt., Engrs., N. A. Discharged Dec. 13, 1918. E. O. T. S., Camp Lee; Office, Chf. of Engrs., Washington, D. C.; Co. Comdr., 211th Engrs., Sapper Regt., 11th Div.

LAND, JOHN THOMAS
Entered service Dec. 28, 1917; Capt., Engrs., N. A., July 30, 1917. Discharged Mar. 28, 1918.

LANE, ALBERT LOSSEN
Entered service Aug. 26, 1917; 1st Lt., Inf., Nov. 26, 1917; Capt., Inf., Sept. 5, 1918. With 50th Inf. training troops; Co. Comdr., R. O. T. C. and S. A. T. C., Plattsburg; college detail instructing engr. units.

LANE, EMORY WILSON
Entered service Apr. 1, 1918; 2d Lt., A. S., U. S. A., Sept. 21, 1918. Discharged Dec. 21, 1918. Eng. Dept., Dorr Field; Asst. Supt. Constr., A. S. Mechanics Training School, St. Paul, Minn.

LANSDALE, JOHN
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 28, 1917; Maj., Engrs., U. S. A., Aug. 26, 1918. Overseas service Dec. 18, 1917–July 6, 1919. Discharged July 30, 1919. Engr. in chg. railroad to American Hospital at Angers, France; Supt., R. R. Constr., Nevers, France; Chf. Asst. to Chf. of Railroad and Dock Constr., Office Chf. Engr., A. E. F.; Chf. of Section, Tours, France. Citation by Gen. Pershing; Officier de Academie, with Palms; Chevalier du Merite Agricole.

LARKIN, CHARLES RAYMOND
Entered service Sept. 12, 1918, as Pvt., Q. M. C., U. S. A. Constr. Div., U. S. A., Dec. 6, 1918. Discharged Dec. 8, 1918.

LaROCHE, ARTHUR LEWIS
1st Lt., Engrs., U. S. A., A. E. F.*

LARRISON, GEORGE KIRKPATRICK
Entered service Oct. 4, 1917; Capt., Engr. R. C., July 15, 1917. Discharged Dec. 31, 1919. R. O. T. C., Schofield Barracks, Hawaii; Asst. to Dept. Engr., Hawaiian Dept.; Gen. Staff.

LATHBURY, BENJAMIN BRENTNALL
Entered service Nov. 6, 1917, as Maj., Ord. Dept., U. S. A.; Lt. Col., Ord. Dept., U. S. A., Oct. 5, 1918; Col., Q. M. C., U. S. A., July 7, 1919. In office Chf. of Ord. Dept.; representative of Chf. of Ord. in Constr. Div.; on Secretary of War's Special Committee for Settlement of Certain Contracts Involving Constr. of Plant Facilities; on Special Committee for Settlement of Contracts with Canadians; Asst. to the Asst. Secy. of War.

LATIMER, CLAUDE ALFRED
Entered service May 8, 1917; 2d Lt., E. O. R. C., Jan. 15, 1917; 1st Lt., Engrs., N. A., July 20, 1918. Discharged Dec. 20, 1918. Training Camp, Ft. Oglethorpe, Ga.; with 3d Engrs. in Panama, in chg. constr. military roads and trails.

LAVERTY, SAMUEL PERRY
Entered service Sept. 10, 1917, as Pvt., Inf., N. A.; Reg. Sup. Sgt., Ammunition Tr., Oct. 18, 1917; 2d Lt., Engrs., N. A., Mar. 31, 1918; 1st. Lt., Engrs., N. A., June 22, 1918. Overseas service Aug. 30, 1918–June, 1919. Discharged July 11, 1919. With 364th Inf., 316th Ammunition Troops, 522d Engrs.; Adj., 522d Engrs., Meuse-Argonne offensive; Toul Sector; Regtl. Historian; Gas Officer; Advance Officer at Port of Embarkation. Two stars.

LAWRENCE, FRANK ELMAKER
Entered service May 8, 1917; 1st Lt., Engrs., N. A., July 21, 1917; Capt., Engrs., N. A., Mar. 10, 1918; Maj., Engrs., U. S. A., Feb. 13, 1919. Overseas service July 28, 1917–July 13, 1919. Discharged July 14, 1919. With 17th Engrs.; represented American Relief Administration in Jugo-Slavia at Ragusa, Dalmatia, Belgrade, Serbia. Class III, Order of St. Sava, Serbia.

LAWTON, RICHARD MACK
Maj., Engrs., U. S. A., A. E. F.*

LEA, SUMTER, JR.
Capt., Engrs., U. S. A., A. E. F.*

LEACH, HARRY RAYMOND
Sgt., 1st Class, Sig. C., U. S. A.*

LEARNED, ALBERT PREISACH
Entered service Mar. 26, 1918, as 1st Lt., San. C., U. S. A. Discharged Feb. 20, 1919. C. O. Detachment, and Camp. San. Engr., Camp Beauregard.

LeBARON, ROBERT WENDELL PHILLIPS
Entered service May 8, 1917, as 2d Lt., E. O. R. C.; 1st Lt., E. O. R. C., July 10, 1917. Overseas service July 23, 1917–July 17, 1919. Discharged Aug. 2, 1919. With 13th Engrs.; Military Instr.; attached to French 2d and 4th Armies; Champagne-Marne and Meuse-Argonne offensives. Two stars.

LECKIE, ALEXANDER ROSS
Entered service June 20, 1917; Capt., E. O. R. C., Aug. 5, 1917. Overseas service July 30, 1918–May 30, 1919. Discharged June 24, 1919. C. O., Co. B., 111th Engrs.; Topographical Officer, 111th Engrs.; 1st Army Corps, St. Mihiel and Meuse-Argonne offensives. Two stars.

LEE, ALONZO CHURCH
Entered service Aug. 27, 1917, 2d Officers' Training Camp; 1st Lt., Inf., N. A., Nov. 27th, 1917. Overseas service July 7, 1918–July 17, 1919. Discharged Aug. 6, 1919. With 6th Div., 51st Inf., and 17th M. G. Bn.; Bn. Supply Officer; Vosges Mountains-Gerardmere Sector and Meuse-Argonne offensives. Two stars.

LEE, AUGUSTINE LEFTWICH
Entered service Dec. 27, 1917; Capt., Engrs., N. A., Sept. 27, 1917; Maj., Engrs., U. S. A., Aug. 9, 1918. Overseas service Oct. 27, 1918–July 10, 1919. Discharged July 18, 1919. Co. Comdr., 522d Engrs.; Musical Director, Service Battalions, Camp Humphreys; with 540th Engrs. on constr. Rifle Range; salvaging German, French and American Rys. at St. Mihiel and Verdun; Supt. and Sec. Engr. of Roads, Base Sec. No. 7; Asst. Engr., Base Sec. No. 2; C. O., 130th Engr. Bn.

LEE, CHARLES AVERY
Entered service Apr. 16, 1918, as Lt., C. E. C., U. S. N.; Lt. Comdr., C. E. C., U. S. N., June, 1919. Released from active duty June 23, 1920. With Bureau of Yards and Docks, Washington, D. C.; Public Works Dept., Philadelphia Navy Yard, in chg. constr. of dry dock.

LEE, CHARLES HAMILTON
Entered service May 8, 1917; 2d Lt., E. O. R. C., Mar. 1, 1917; 1st Lt., E. O. R. C., Sept. 8, 1917; Capt., Engrs., U. S. A., Oct. 31, 1918. Overseas service Oct. 18, 1917–Mar. 12, 1919. Discharged July 21, 1919. With 26th Engrs.; Water Supply Officer; Water Intelligence Officer, Water Supply Service, 1st Army; St. Mihiel and Meuse-Argonne offensives. Three stars.

LEE, JOHN LOUIS
Entered service Jan. 3, 1918, as Maj., Q. M. C., N. A. Discharged July 1, 1919. Const. Q. M., Charleston and New Orleans; Member, Ord. Claim Bd., Gillespie Plant Explosion, Perth Amboy, N. J.

LEEDS, CHARLES TILESTON
Entered service June 13, 1899; Capt., C. of E., U. S. A., Feb. 27, 1911; retired Sept. 28, 1912; re-entered service Apr. 7, 1917; Maj., Engrs., U. S. A., Oct. 3, 1918. Released from active duty July 10, 1919. Dist. Engr., Los Angeles, Cal., on constr. of fortifications and improvement of harbors; C. O., S. A. T. C., Throop College of Technology.

LEFEVER, KENNETH W.
Entered service Aug. 5, 1918, as Pvt., Engrs., N. A. Overseas service Sept. 15, 1918–June 19, 1919. Discharged July 3, 1919. With 113th Engrs., on hosp. and barrack constr. and road repairing.

LEHRBACH, HENRY GARDNER
Entered service June 28, 1917; Lt., Jr. Grade, C. E. C., U. S. N., June 15, 1917; Lt., C. E. C., U. S. N., Oct. 15, 1917. Asst. Public Works Officer, Navy Yard, Charleston, S. C., superv. constr.

LEISEN, THEODORE ALFRED
Entered service Oct. 27, 1917, as Maj., Q. M. C., Constr. Div., N. A.; Lt. Col., Q. M. C., Constr. Div., June, 1919. Discharged June 6, 1919. In chg. Utilities, Camp Custer; Const. Q. M., extension to Camp Custer.

LEISER, FERDINAND
Candidate, C. O. T. S., F. A., U. S. A.*

LELAND, ORA MINER
Entered service Apr. 16, 1917, as Capt., E. O. R. C.; Maj., E. O. R. C., Aug. 14, 1917; Lt. Col., Engr. R. C., Apr. 17, 1918. Overseas service May 27, 1917–May 26, 1919. Discharged June 19, 1919. With 314th Engrs., 89th Div., Commandant, Technical School, 89th Div. Three stars.

LEMEN, WILLIAM CASWELL SMITH
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 21, 1917; Maj., Engrs., N. A., July 15, 1918; Lt. Col., Engrs., U. S. A., Oct. 24, 1918. Overseas service July 28, 1919–Sept. 17, 1919. Discharged Oct. 4, 1919. Special duty at Port of Embarkation; in chg. constr. and operation Engr. Depot, Kearny, N. J.; Engr. Officer, Port of Embarkation; Executive Officer, Port Storage, Port of New York.

LEONARD, OLIVER YEATON
Entered service Jan. 31, 1918, as 1st Lt., Eng. Div., Ord. Dept., U. S. A. Discharged July 23, 1919. Gen. design, Washington, D. C.; transferred to Salvage Bd. at Toronto, Canada, checking claims against U. S. Govt.; estimating cost of buildings.

LETTON, HARRY PIKE
Entered service Sept. 2, 1917; Capt., E. O. R. C., Apr. 2, 1917. Overseas service Jan. 23, 1918–Mar. 19, 1919. Discharged Mar. 30, 1919. With 111th Engrs. at Camp Bowie; Water Supply Service, A. E. F., in chg. quality of water at Base Sec. No. 2 and with 1st Army. Torpedoed off North coast of Ireland on S. S. *Tuscania*.

LEVY, AARON GRETZNER
Entered service Aug. 19, 1918, as Capt., Q. M. C., Constr. Div. Discharged Apr. 15, 1919. Asst. Utilities Officer in chg. Water, Sewer and Pumping Secs., Camp MacArthur; Executive Officer of Utilities; details on salvaging of camp.

LEWIS, CHARLES GRANVILLE
Entered service Sept. 18, 1917, as Pvt., Engrs., N. A.; Cpl. Engrs., N. A., Mar. 28, 1918; M. E., Jr. Grade, Engrs., U. S. A., Nov. 26, 1918. Overseas service Dec. 26, 1917–Aug., 1919. Discharged Sept. 3, 1919. With 21st Engrs., reconnaissance and location light ry.; with North Russian Expedition on Murmansk-Petrograd Railroad. Three stars.

LEWIS, CHESTER BROOKS
Entered service Aug. 28, 1918; 1st Lt., Engrs., U. S. A., Sept. 4, 1918. Discharged Feb. 5, 1919. With 564th Engr. Service Bn.; Co. Comdr., 217th Engrs., Camp Beauregard.

LEWIS, FREDERICK HUMPHREVILLE
Entered service Dec. 28, 1917; Maj., E. O. R. C., June 17, 1917. Discharged Dec. 5, 1918. On staff, Chf. of Engrs., Washington, D. C.; with 4th Rgt., Engr. Replacement Troops, Camp Humphreys.

LEWIS, HAROLD MacLEAN
Entered service May 14, 1917; 2d Lt., E. O. R. C., June 11, 1917; 1st Lt., Engrs., U. S. A., Oct. 14, 1918; Capt., Engrs., U. S. A., Apr. 7, 1919. Overseas service Nov. 12, 1917–June 22, 1919. Discharged June 30, 1919. Adj. and Co. Comdr., 20th and 502d Engrs.

LEWIS, LUTHER HAMMOND
Entered service Apr. 13, 1918; Maj., Q. M. C., Constr. Div., N. A., Apr. 11, 1918. Discharged May 7, 1919. Liaison Officer between Constr. Div. and Surgeon General's Office on hosp. constr. in the U. S.; Const. Q. M. in chg. of all hosp. constr., except Base Hosps., at camps in U. S.

LIBBEY, VALENTINE BROUSSEAU
Entered service May 11, 1917; 2d Lt., E. O. R. C., Aug. 13, 1917; 1st Lt., Engrs., U. S. A., Sept. 17, 1918. Overseas service, Sept. 10, 1917–Sept. 17, 1918. Discharged Nov. 29, 1918. Am. Engr. School, Ecole du Genie, Versailles; with 117th Engrs.; with 4th Engr. Training Regt. Three stars.

LIGHTNER, GEORGE W. CASS
Entered service May 14, 1917; 1st Lt., Engrs., N. A., Sept. 8, 1917; Capt., Engrs., N. A., Feb. 25, 1918. Overseas service Dec. 26, 1917–May 22, 1919. Discharged May 24, 1919. With 21st Engrs. as Regtl. Supply Officer; Army School of the Line, and Army Gen. Staff College, A. E. F.; 2d Army, Advance Sec., S. O. S., A. E. F.

LILLY, RIDGELY CASEY
Entered service May 11, 1917; 1st Lt., E. O. R. C., June 13, 1917; Capt., Engrs., U. S. A., Aug. 29, 1918. Overseas service Sept. 12, 1917–Aug. 8, 1919. Discharged Aug. 29, 1919. Bn. Adj., 312th and 2d Engrs. Five stars.

LINDBERY, CHARLES ARTHUR
Entered service Dec. 19, 1917; Capt., E. O. R. C., Sept. 21, 1917. E. O. T. C., Camp Lee; with 601st Engrs., Camp Humphreys; student, E. O. T. C., Camps Lee and Humphreys. Died May 27, 1918, in active service at Camp Lee, Va.

LINDHE, JOHN BIRGER
Entered service Mar. 23, 1917; Lt., U. S. N. R. F., Nov. 22, 1917; Lt. Comdr., U. S. N., May, 1918. Overseas service Nov. 22, 1917–Aug. 2, 1919. Released from active service Aug. 2, 1919. Citation for meritorious service. One star.

LINEBERGER, WALTER FRANKLIN
Entered service June 19, 1917, as Capt., E. O. R. C.; Maj., Engrs., U. S. A., Feb. 13, 1919. Overseas service Dec. 31, 1917–Mar. 29, 1919. Discharged Mar. 29, 1919. With 116th Engrs., 1st Engrs., 107th Engrs., 115th Engrs., Combat Units. Croix de Guerre; Citation French IV Army. Five stars.

LINSLEY, CHARLES WELLS
Entered service Apr. 8, 1918; Lt., Jr. Grade, C. E. C., U. S. N. R. F., Apr. 1, 1918; Lt., C. E. C., U. S. N. R. F., Feb. 25, 1919. Released from active service Apr. 8, 1919. Asst. to Public Works Officer, Hampton Roads, Va., constr. of training stas., operating base and air sta.

LIPARI, ATTILIO FELIX
Entered service Sept. 10, 1917, as Pvt. Sig. C., Aviation Sec.; 2d Lt., A. S., Oct. 4, 1918. Discharged Dec. 20, 1918. Assigned to duty with the Italian Aviation Mission, Washington, D. C.

LISMAN, OLIVER CROMWELL
1st Lt., Engrs., U. S. A., A. E. F.*

LIVERMORE, NORMAN BANKS
Entered service Sept 2, 1917; Capt., Engrs., N. A., Oct. 15, 1917; Maj., M. T. C., N. A., Aug., 1918. Overseas service Oct. 20, 1917–Jan. 10, 1919. Discharged Jan. 13, 1919. With British Army in Belgium and with A. E. F. in France. Officier, Legion d'Honneur.

LIVINGSTON, ARCHIBALD ROGERS
Entered service Aug. 5, 1917, as Capt., Inf., U. S. A.; Maj., C. of E., U. S. A., Oct. 13, 1917. Overseas service Aug. 8, 1918–June 28, 1919. Discharged July 11, 1919. With 115th Engrs. in Marbache Sector, north of Toul, and in Coblenz, Army of Occupation. One star.

LOCKHARDT, WILLIAM FRANCIS
Entered service Dec. 15, 1917, as Sgt., 1st Class, Sig. C., A. S., U. S. A.; 2d Lt., Q. M. C., Constr. Div., N. A., July 11, 1918; 1st Lt., Q. M. C., Constr. Div., U. S. A., Aug. 24, 1918. Discharged Feb. 17, 1919. Asst. Supt. and Supt., Aviation and Supply Warehouse, Middletown, Pa, and Aviation Repair Depot, Indianapolis, Ind.; Staff Officer with Const. Q. M., Army Supply Base, Brooklyn, N. Y.

LOCKWOOD, RICHARD JOHN
Entered service Aug. 26, 1917; 2d Lt., F. A., N. A., Nov. 27, 1917; Capt., Engrs., N. A., Jan. 7, 1918; Maj., Engrs., U. S. A., Nov. 7, 1918. Overseas service Apr. 11, 1918–June 27, 1919. Discharged July 3, 1919. With 334th F. A., 87th Div., Div. Light Rys. and Roads, 1st and 3d Armies, A. E. F.; Chf. Engr., Light Rys., A. E. F. Two stars.

LOGAN, CHESTER RUSSEL
Entered service May 15, 1917; Capt., E. O. R. C., Aug. 15, 1917. Overseas service Nov. 12, 1917–June 11, 1919. Discharged Oct. 28, 1919. In chg. constr. at Valdahon; Bn. Adj., 318th Engrs., Vosges and Meuse-Argonne operations; with Peace Commission, War Damages Sec. Two stars.

LOGAN, VERNON LEMLEY
Entered service May 14, 1917, as Pvt., Engrs., Kansas N. G.; Sgt., 1st Class, Engrs., N. A., Aug. 1, 1917; 2d Lt., Engrs., N. A., Apr. 1, 1918; 1st Lt., Engrs., N. A., Aug. 1, 1918. Overseas service July 29, 1918–July 5, 1919. Discharged July 15, 1919. Constr. work at Camp Funston; Co. Comdr., 530th Engrs., 1st Army, A. E. F.; served at St. Mihiel and Argonne with 2d and 3d Armies; Advance Sec., S. O. S., A. E. F. Two stars.

LONGLEY, FRANCIS FIELDING
Col., Engrs., U. S. A., A. E. F.*

LORD, ARTHUR RUSSELL
Entered service Sept. 2, 1917; Capt., Engrs., N. A., Aug. 1, 1917; Maj. Engrs., N. A., July 23, 1918. Discharged May 17, 1919. With 108th and 513th Engrs.; Concrete Ship Sec., U. S. Shipping Board, on research investigations.

LOSH, ALBERT RICHARD
Entered service Oct. 17, 1918, as Capt., C. W. S., U. S. A. Discharged Jan. 15, 1919. Plant constr., Edgewood Arsenal.

LOUCKES, FRANK IRWIN
Entered service May 5, 1917, as Capt., E. O. R. C.; Maj., T. C., U. S. A., Feb. 14, 1919. Overseas service Jan. 26, 1918–July 12, 1919. Discharged July 29, 1919. In chg. barge repair yards at Epone and St. Mammes, France.

LOUGHRAN, JAMES FRANCIS
Entered service Aug. 30, 1918, as Lt., C. E. C., U. S. N. R. F. Released from active service Apr. 9, 1919. Transportation Mgr., Naval Operating Base, Hampton Roads, Va.

LOVERING, HARRY DOUGLAS
Entered service Dec. 28, 1917; 1st Lt., E. O. R. C., Sept. 13, 1917; Capt., Engrs., U. S. A., Aug. 3, 1918. Overseas service Feb. 19, 1918–Aug. 3, 1918. Discharged Jan. 18, 1919. With 107th Engrs., A. E. F.; Co. Comdr., 3d Engrs., Camp Humphreys.

LOWE-BROWN, WILLIAM LOWE
Entered service June 9, 1917, as Lt. Col., Royal Engrs., England; Lt. Col., Royal Marine Engrs., Mar. 18, 1918. Released from active service Dec. 27, 1918. C. O., Constr. Train Ferry, Richborough; Admiralty Works, Southwick, England.

LUCAS, EUGENE WILLETT VAN COURT †
Lt. Col., Engrs., U. S. A.*

LUIGGI, LUIGI
Entered service Aug., 1915, as Col., Artillery Corps, Italian Army. Released from active duty Dec. 20, 1918. Member of Committee for Munition Works. Gold medal from Minister of Munitions, Italy, for distinguished service.

LUMSDEN, HUGH JOHN
Entered service Apr. 6, 1918, as Capt., Q. M. C., Constr. Div.; Maj., Q. M. C., Constr. Div., U. S. A., July 23, 1919. Asst. Superv. Const. Officer, Washington, D. C.; Const. Officer, Edgewood and Frankford Arsenals.

LUND, ALFRED MAJENDIE
Entered service Feb. 23, 1918, as Maj., San. C., N. A. Discharged Feb. 8, 1919. San. Engr., Camp Humphreys and Camp Hancock.

LUNDGREN, LEONARD
Entered service Aug. 5, 1917, as Capt., Engrs., N. A. Overseas service Nov. 26, 1917–July 14, 1919. Discharged Nov. 24, 1919. Army General Staff College, A. E. F., Sr. Instr., Eng. Army School of the Line. Citation for exceptionally meritorious and conspicuous service. One star.

LYNCH, EDWARD MELVILLE
Entered service Nov. 2, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 3, 1918.

LYNCH, JOHN FRANCIS
Capt., Engrs., U. S. A.*

LYON, GEORGE JOHN
Entered service Aug. 3, 1917; Capt., E. O. R. C., June 19, 1917. Discharged Aug. 17, 1920. With Constr. Div., Camp Hancock; Schenectady Q. M. Depot, Water-Works Constr., Hampton Roads, Va.

LYON, LÉON ÉLIE
Entered service Jan. 19, 1917, as Capt., E. O. R. C.; Maj., R. T. C., June 17, 1918; Lt. Col., R. T. C., Nov. 2, 1918. Overseas service Oct. 3, 1917–July 28, 1919. Discharged Oct. 15, 1919. Chf. of Inland Waterway Transport Div., A. E. F. Citation from Gen. Pershing for exceptionally meritorious and conspicuous service. Officier d'Académie; Order of Univ. Palms.

LYON, WARREN ADAMS
Entered service Oct. 16, 1917, as 2d Lt., C. of E., U. S. A.; 1st Lt., C. of E., U. S. A., Feb., 1918; Capt., C. of E., U. S. A., May 1918. Overseas service Apr. 30, 1918–Nov. 18, 1918. With 4th Engrs. (Sapper Regt.), through Aisne-Marne, St. Mihiel and Meuse-Argonne offensives. Four stars.

LYONS, HAROLD CHANDOS
Entered service May 12, 1917; 1st Lt., Engr. R. C., Aug. 15, 1917; Capt. Engrs., U. S. A., Aug. 10, 1918. Overseas service Nov. 12, 1917–Aug. 20, 1919. Discharged Sept. 12, 1919. With 20th Engrs., A. E. F. Citation from Gen. Pershing for conspicuous conduct.

MACARTNEY, MORTON
Entered service June 13, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged Jan. 16, 1919. Served at Washington, D. C. and as Utility Officer, Camp Humphreys.

MacGLASHAN, ALEXANDER
Entered service Apr. 12, 1917; Maj., Inf., U. S. A., Aug. 18, 1916; Maj., Engrs., N. A., Oct. 4, 1917. Overseas service June 18, 1918–May 24, 1919. Discharged May 28, 1919. With 1st Army, Meuse-Argonne offensive; with 104th Engrs. Citation for work on constr. of bridges. Two stars.

MACHEN, HENRY BENNETT
Entered service July 1st, 1918, as Maj., Ord. Dept., U. S. A. Discharged Sept. 30, 1920. In chg. constr., By-Product Coke Oven Plant, Buffalo, N. Y.

MACINTOSH, PERCY HUGH MARSHALL
Capt., Tropical Force, Australia, B. E. F.*

MACK, PAUL WARDLAW
Capt., Engrs., U. S. A.*

MACKENZIE, ALEXANDER
Maj. Gen., U. S. A. (*Retired*); returned to active service May, 1917. Released from active duty May, 1919. River and harbor work as Div. Engr., Northwestern Div., and Dist. Engr. on upper Mississippi River Improvement, releasing younger officers for overseas service.

MacKENZIE, LEON RODERICK
Entered service July 18, 1918, as Capt., Q. M. C., Constr. Div., N. A. Discharged Feb. 14, 1919. Co. and Bn. Comdr., in chg. of training and equipment; Engr. of Roads, Gen. Staff, Newport News, Va.

MacNAUGHTON, PERCIVAL JOHN
Entered service May 21, 1918, as Ensign, C. E. C., U. S. N.; Lt., Junior Grade, C. E. C., U. S. N., Dec. 21, 1918. Released from active duty Dec. 28, 1918. With Bureau of Yards and Docks, Philadelphia Yard; Officer in chg. estimates, vouchers and contracts on constr. work, Structural Eng. Dept.

MACOMB, JOHN DE NAVARRE
Entered service Sept. 2, 1917; Capt., Engrs., E. O. R. C., June 23, 1917; Maj., Engrs., N. A., July 30, 1918. Overseas service Jan. 10, 1918–Aug. 20, 1918. Discharged Jan. 4, 1919. In

† Died Mar. 8, 1921.

chg., track constr., Storage Depot, Gievres, France; C. O., 548th Engrs., Camp Humphreys, Receiving Officer, Fort Benjamin Harrison; attached to 20th Engrs., Camp Forrest.

McCALLA, WILLIAM AUGUSTUS
Entered service Sept. 12, 1917; Maj., E. O. R. C., June 19, 1917; Lt. Col., Engrs., U. S. A., Feb. 14, 1919. Overseas service Nov. 26, 1917–Apr. 20, 1919. Discharged Apr. 24, 1919. Bn. Comdr., 504th Engrs.; in chg., constr. of A. S. Production, France; in chg., highway repair. Chevalier, Legion d'Honneur.

McCANDLISS, EDGAR SCOTT
Entered service May 15, 1917; Capt., E. O. R. C., July 10, 1917. Overseas service June 11, 1918–Jan. 22, 1919. Discharged Jan. 27, 1919. Adj., 1st Bn., 314th Engrs.; Bn. Comdr., St. Mihiel offensive. Two stars.

McCANDLISS, LESTER CHIPMAN
Entered service Apr., 1917, as 1st Lt., E. O. R. C.; Capt., Engr. R. C., Dec., 1917. Overseas service July 9, 1917–Apr. 8, 1919. Discharged May 1, 1919. Co. Comdr., 15th Engrs.; Army Gen. Staff College; constr., Advance Sec., S. O. S., A. E. F.

McCLEAN, GEORGE THOMAS
Entered service June, 1917, as Capt., E. O. R. C. Overseas service June 23, 1918–Apr. 12, 1919. Discharged Apr. 28, 1919. With 29th Engrs.; in chg., Technical Sec., Engr. Purchasing Office.

McCLELLAN, GEORGE ABRAHAM
Entered service Feb. 2, 1918, as Pvt., Sig. R. C., A. S.; 2d Lt., Sig. R. C., A. S., May 24, 1918; 2d Lt., Engrs., U. S. A., Aug. 29, 1918. Discharged Dec. 11, 1918. In chg., Aero Repair and Machine Shops, Chanute Field; with 9th Engrs.

McCLINTOCK, HALLETT EDWARD
Entered service Sept. 2, 1917; Capt., Engr. R. C., Aug. 1, 1917. Overseas service Jan. 26, 1918–Jan. 5, 1919. Discharged Jan. 9, 1919. With 112th Engrs.; special and reconnaissance work, Transportation Sec., A. E. F.; Supply Officer, Railroad and Dock Sec., Div. of Constr. and Forestry, A. E. F.

McCLURE, HARRY CLIFFORD
Entered service June 21, 1918, as Capt., Ord. Dept., U. S. A. Discharged Mar. 6, 1919. With Nitrate Div. investigating nitrate plant sites; Ord. Officer and C. O. at U. S. Nitrate Plant No. 4, Cincinnati, Ohio; supervising work on roads, sewers, etc.

McCLURE, HUNTER
Entered service Dec., 1917, as 1st Lt., E. O. R. C. Overseas service, 1918. E. O. T. C., Camp Lee; with 21st Engrs. at Camp Humphreys and in France. Died Sept. 26, 1918, in active service in France.

McCOMB, DANA QUICK
Entered service July 13, 1917, as Capt., E. O. R. C.; Maj., Engr. R. C., June 19, 1918. Overseas service Sept. 5, 1917–July 19, 1919. Discharged Aug. 13, 1919. With 3d Engrs. as Asst. to Dist. Engr. and Engr. Officer in chg. defensive work, constr. and maintenance of roads and fortifications; Asst. to Coast Defense Q. M., serving in Philippine Islands.

McCONNELL, EDWARD HERBERT
Entered service Sept. 2, 1917; 1st Lt., Engr. R. C., Aug. 1, 1917; Capt., Engrs., U. S. A., Apr. 13, 1919. Overseas service May 10, 1918–July 12, 1919. Discharged Aug. 3, 1919. With 128th Engrs.; 1st Army Artillery and Railway Constr. Bn., Bridge Sec., A. E. F.

McCRONE, ROSSITER MAGERS
Entered service July 6, 1918, as 1st Lt., Engrs., N. A. Discharged Jan. 20, 1919. In training, 6th, 10th, and 3d Engr. Training Regts.

McCULLOUGH, CONDE BALCOM
Capt., Engrs., U. S. A.*

McCULLOUGH, ERNEST
Entered service May 31, 1917; Maj., E. O. R. C., May 4, 1917; Maj., C. W. S., U. S. A., Sept. 15, 1918; Lt. Col., C. W. S., U. S. A., Feb. 14, 1919. Overseas service June 2, 1917–Aug. 26, 1919. Discharged July 24, 1920. Chf. Engr., American Red Cross; Lecturer on Gas Warfare, Army Engineer School, Langres; Chf. Gas Officer on staff of Maj. Gen. Liggett, 1st Army Corps; Chf. Gas Officer, on staff of Maj. Gen. McGlachlin, 1st Army Artillery; Asst. Chf. and Chf., Artillery Sec., Offense Div., C. W. S.; Constr. Engr., Rents, Reclamations and Claims Service; Asst. Commandant, Lakehurst Proving Ground and Director, Service School, C. W. S. Two stars. One wound.

McDERMITH, ORO
Entered service Dec. 28, 1917; Capt., Engrs., N. A., Sept. 27, 1917. Overseas service June 10, 1918–May 23, 1919. Discharged June 23, 1919. Co. Comdr., 104th Engrs.; Topographical Officer, 29th Div. One star.

McDONALD, HARRY L.
Capt., Engrs., U. S. A.*

McDONALD, WILLIAM NAYLOR
Entered service July 8, 1918, as Capt., Engrs., N. A. Discharged Dec. 19, 1919.

McDONNELL, FRANCIS REGIS
Entered service June 28, 1918, as Lt., Jr. Grade, U. S. N. R. F.; Lt., U. S. N. R. F., Jan. 1, 1919. Overseas service July 31, 1918–June 10, 1919. Public Works Officer, Aviation Repair Base, Eastleigh, England; Executive Officer, Relief Unit, Lille, France; Naval Aviator at Naval Air Stas., Akron, Ohio, and Pensacola, Fla. One star.

McDONOUGH, MICHAEL JOSEPH†
Entered service Feb. 15, 1899; Maj., C. of E., U. S. A., at declaration of war; Col., Engrs., N. A., Aug. 5, 1917. Instr., Plattsburg Camp; C. O., 114th Engrs., Camp Beauregarde.

McENTIRE, LLOYD
Pvt., Inf., U. S. A.*

McFARLAND, HARRY FONTAINE, JR.
Entered service June 18, 1917, as 1st Lt., E. O. R. C.; Capt., Engr. R. C., Apr. 2, 1918. Overseas service July 26, 1917–Apr. 28, 1919. Discharged June 3, 1919. With 12th Engrs. and various British Armies. Military Cross, Great Britain. Nine stars.

McGEE, ARTHUR BRANCH
Entered service Jan. 25, 1918, as Pvt., Ord. Dept., Nitrate Div.; Sgt., 1st Class, Ord. Dept., Nitrate Div., Mar. 4, 1918. Discharged Mar. 20, 1919. On detached service with Alabama Power Co., Birmingham, Ala.

McGEE, HAROLD GILBERT
Entered service Feb. 13, 1918, as 1st Lt., San. C., U. S. A.; Capt., San. C., U. S. A., Sept. 3, 1918. Discharged Apr. 9, 1919. Camp Sanitary Engr., Camps Jackson and Dodge.

McGEE, ROGER KEYES
Entered service June 4, 1918, as Capt., Engrs., N. A. Overseas service Oct. 25, 1918–June 12, 1919. Discharged Aug. 7, 1919. Co. Comdr. and C. O., 547th Engrs.

McGEEHAN, PAUL
Entered service May 1, 1917, as Capt., E. O. R. C. Overseas service July 28, 1917–Apr. 26, 1919. Discharged Oct. 25, 1919. With 12th Engrs. on light ry. work; one year on British Front, battles of Cambrai and the Somme; with American Army at St. Mihiel and Toul. Four stars.

McGILVRAY, THOMAS FORRESTER.
Entered service Apr. 18, 1917, as Maj., E. O. R. C. Discharged Feb. 26, 1918.

McGLATHERY, SAM LYON
Capt., Engrs., U. S. A., A. E. F.*

McGREGOR, ROBERT ROY
Entered service May, 1917; Capt., Engrs., N. A., Sept., 1917; Maj., Engrs., N. A., July, 1918. Overseas service Dec., 1917–Aug., 1918. Discharged Dec., 1918. Co. Comdr. 21st Engrs., Camp Grant and in France; with 1st Div. in Lorraine; const. light rys. in Advance Sector, St. Mihiel and Pont a Mousson.

McGREW, JOHN ALEXANDER
Entered service July, 1917; Maj., Q. M. R. C., Dec. 8, 1916; Maj., Engrs., U. S. A., Aug., 1918. Overseas service June, 1918–Jan., 1919. Discharged Feb. 21, 1919. Transportation Officer, Camp Upton; in chg. pier and ordnance depot constr., Portsmouth, Va.; Terminal Supt., Bordeaux, France.

McINTOSH, SAMUEL FRASER
Entered service Jan. 31, 1918, as Capt., Ord. Dept. Discharged Feb. 18, 1919. In chg. design ordnance shipping depot and mechanical trades in constr. Proving Grounds, Aberdeen, Md.

McINTYRE, LEWIS WEDSEL
Entered service June 15, 1918, as Pvt., C. A. C., U. S. A.; 2d Lt., C. A. C., U. S. A., Sept., 1918. Discharged Sept., 1919. Instr., Orientation Dept., Coast Artillery School.

McKAY, GEORGE ALBERT
Entered service May 26, 1902; through all grades in C. E. C., U. S. N., to Comdr., Aug. 1, 1917. With Bureau of Yards and Docks, Washington, D. C.

McKENNEY, CHARLES ALBERT
Entered service May 16, 1917; Maj., Engrs., N. A., July 26, 1917; Lt. Col., Engr. R. C., Mar. 20, 1918; Col., Engrs., U. S. A., Sept. 29, 1918. Discharged Oct. 25, 1919. Prin. Asst. to Chf. of Engrs. in chg. of supplies; representative Priorities Div., War Industries Bd., in chg. of production, transportation, etc., of all supplies; Cht., Priority Sec., Purchase, Storage and Traffic Div.; Asst. Dir. of Purchases, in chg. of organization of war claim settlements; Member, Claims Bd. of War Dept.

McKINSTRY, CHARLES HEDGES
Entered service Sept., 1884; through all grades C. of E., U. S. A., to Brig. Gen., Aug. 5, 1917. Overseas service July 14, 1917–Aug. 6, 1919. Retired from active service Sept. 16, 1919. C. O., 11th Engrs., in command 1st F. A. Brig.; Director of Light Rys. and Roads. Officier, Legion d'Honneur; citation for meritorious service. Three stars.

McLANE, GLENWOOD LYLE
Entered service July 7, 1917, as Capt., E. O. R. C.; Acting Maj., Engrs., U. S. A., Mar. 14, 1919. Overseas service May 2, 1918–June 27, 1919. Discharged July 23, 1919. With 110th and 546th Engrs.; Asst. Engr. of Roads in technical chg. German prisoners of war on road labor, 1st Army, A. E. F. One star.

McLOUD, PAUL
Entered service May 28, 1917; 1st Lt., E. O. R. C., June 13, 1917; Capt., Engrs., U. S. A., Aug. 31, 1918; Maj., Engrs., U. S. A., Apr. 8, 1919. Overseas service July 12, 1917–Oct. 2, 1919. Discharged Oct. 2, 1919. With 11th Engrs. on railroad, harbor, hosp. and camp projects, S. O. S.; supervising of allocation duty, Base No. 5. Citation for conspicuous bravery by Sir Douglas Haig. British Military Cross; Distinguished Service Cross. Two stars.

† Died Feb. 13, 1921.

McMEEKIN, CHARLES WILLIAM
Entered service July 18, 1917, as Maj., Engr. R. C. Released from active duty Oct. 27, 1919. War Plans Div., Gen. Staff, Chf. of Inventions Section.

McMILLAN, W. BRUCE
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., July 10, 1917. Overseas service Jan. 27, 1918–Apr. 5, 1919. Discharged Apr. 21, 1919. With 101st Engrs.; Instr., 3d Corps Schools.

McMULLEN, CLEMENTS
Entered service Sept. 1, 1917; 2d Lt., A. S., U. S. A., Mar. 5, 1918. Instr. in combat flying and aerial gunnery; in chg. Instruction Dept., Aerial Gunnery School.

McNAYR, GEORGE EVERETT
Entered service Sept. 2, 1917; 2d Lt., E. O. R. C., June 24, 1917; 1st Lt., Engrs., N. A., June 5, 1918; Capt., Engrs., U. S. A., May 14, 1919. Overseas service June 29, 1918–July 1, 1919. Discharged July 1, 1919. With 33d Engrs., S. O. S.

McRAE, HENRY CLINTON
Entered service May 12, 1917; 1st Lt., E. O. R. C., June 19, 1917. Overseas service July 10, 1918–Aug. 17, 1918. Discharged Sept. 5, 1919. With 306th, 318th, and 45th Engrs.; Staff of Gen. Supt., T. C., 14th Grand Div., as Supply officer. One star.

MADDOCK, THOMAS
Entered service Feb. 28, 1918, as 1st Lt., Engrs., N. A. Overseas service May 10, 1918–Jan. 31, 1919. Discharged Feb. 6, 1919. With 42d Engrs.; assigned to Staff Comdr., Base Sec. No. 1.

MADDOX, LUTHER ROBINSON
Entered service May 14, 1917; Capt., E. O. R. C., May 2, 1917. Overseas service July 28, 1917–July 29, 1919. Discharged Aug. 26, 1919. With 17th Engrs.; Gen. H. Q., A. E. F., in chg. reports of supply and constr.; Disposition Officer.

MADISON, JAMES TALBOTT
Entered service Sept. 22, 1918, as 1st Lt., Engrs., U. S. A. Discharged May 9, 1919. E. O. T. S., Camp Humphreys.

MAHON, JOHN MONTGOMERY, JR.
Entered service Jan. 4, 1918; Capt., Tank Corps, Dec. 19, 1917. Discharged Dec. 15, 1918. In command 310th Brig. H. Q.; San. Insp., Camp Colt.

MAIL, EUGENE FREDERICK
Entered service May 8, 1917; 2d Lt., E. O. R. C., Mar., 1917; 1st Lt., E. O. R. C., Aug. 15, 1917; Capt., Engrs., U. S. A., May, 1919. Overseas service May 1, 1918–Feb. 2, 1919. Discharged Feb. 12, 1919. Co. Comdr., 4th Engrs. Three stars.

MALLOY, JOHN MICHAEL
Entered service Nov., 1917, as 2d Lt., F. A., U. S. A. Discharged Dec. 20, 1918. Instr., Orientation, Map Work and Map Firing, Camps Funston and Jackson, and Ft. Sill.

MALONE, GEORGE WILSON
Entered service Jan. 5, 1918, R. O. T. S.; Lt., F. A., U. S. A., May, 1918. Overseas service Aug., 1918–Jan., 1919. Discharged Feb. 28, 1919. Intelligence Officer for 145th F. A., 65th Brig.

MALONY, WALDEN LE ROY
Maj., Q. M. C., U. S. A.*

MALSBURY, OMER EVERT
Entered service Sept. 10, 1917; Capt., E. O. R. C., June 13, 1917; Maj., Engrs., N. A., July 20, 1918. Discharged Jan 31, 1919. Co. Comdr., 302d and 3d Engrs.; Topographical Insp., Military Survey of Panama; Engr. Instr., Camp Gaillard, Panama Canal Zone; Adj., 2d Bn., 3d Engrs.; Acting Dist. and Dept. Engr., in chg. of gas training, Panama Canal Dept.; C. O., 2d Bn., 3d Engrs.

MALTBY, FRANK BIERCE
Entered service July 14, 1917; Maj., Engrs., N. A., Feb. 14, 1918; Lt Col., Engrs., U. S. A., Feb. 13, 1919. Overseas service Sept. 20, 1918–June 29, 1919. Discharged July 16, 1919. With Constr. Div. in U. S.; Sec. Engr., Base Sec. No. 1, A. E. F., in chg constr. Officier, Legion d'Honneur.

MANDELZWEIG, HYMAN HENRY
Entered service Apr. 6, 1917; Capt., Engr. R. C., Aug. 23, 1917. Discharged Dec. 1, 1917. Training at Ft. Leavenworth; served in advisory capacity on Draft Boards.

MANDIGO, CLARK ROGERS
Entered service May 8, 1917, as Capt., E. O. R. C.; Maj., Engrs., U. S. A., Sept. 3, 1918. Overseas service June, 1918–Aug. 26, 1918. Discharged Nov. 30, 1918. With 314th Engrs., C. O., 1st Bn., A. E. F.

MANNING, WILLIAM JAMES HENRY
Entered service Sept. 2, 1917; 2d Lt., E. O. R. C., June 20, 1917; 1st Lt., R. T. C., U. S. A., Feb. 24, 1919. Overseas service June 7, 1918–July 4, 1919. Discharged July 17, 1919. With 302d and 52d Engrs.; Co. Comdr. and miscellaneous camp duties; Railroad Transportation Officer, Camps Hunt, Le Courneau, and Gironde, France.

MANSFIELD, NEWTON
Entered service Mar. 29, 1917; Lt. Comdr., U. S. N., July 1, 1907. Released from active service Apr. 21, 1919. Navy Recruiting Officer, Pittsburgh and New York City Districts; Navy Recruiting Insp., Eastern Div.

MARCH, GEORGE MILES
Entered service May 13, 1917; 1st Lt., E. O. R. C., June 28, 1917. Overseas service Feb. 18, 1918–July 5, 1919. Discharged July 11, 1919. With 301st, 314th and 507th Engrs, hosp. constr., road constr. and repair, Beaune, France.

MARKS, EDWIN HALL
Entered service June 15, 1905; through all grades in C. of E., U. S. A., to Col., Engrs., U. S. A., Aug. 1, 1918. Overseas service May 8, 1918–Sept. 12, 1918. Organized 20th, 41st, 42d and 43d Engrs.; in chg. Forestry Sec.; C. O., 20th Engrs. Three stars.

MARRIAN, RALPH RICHARDSON
Entered service 1917, as 2d Lt., Engrs., U. S. A.; 1st Lt., Engrs., U. S. A., 1918. Overseas service 1918. O. T. C., Camp American University; with 105th Engrs., at Camp Sevier and in France. Died Oct. 17, 1918, from wounds received in action near Molain, France.

MARSH, CHARLES REED
Capt., Engrs., U. S. A.*

MARSHALL, ROBERT BRADFORD
Entered service June 22, 1917; Maj., E. O. R. C., Feb. 16, 1917; Lt. Col., Engrs., N. A., July 25, 1918. Discharged Mar. 31, 1919. In chg. military mapping in U. S. for U. S. Geological Survey; organized training school for topographical surveying, Washington, D. C.

MARSTON, ANSON
Entered service July 25, 1917, as Maj., Engrs., N. A.; Lt. Col., Engrs., U. S. A., Sept. 6, 1918. Discharged Dec. 14, 1918. Bn. Comdr., Iowa Engrs., Camp Dodge, on constr. and training, also 109th Engrs., Camp Cody; C. O., 97th Engrs., Camp Leach; in office Chf. of Engrs., outlining curriculum for Engineers School at Camp Humphreys.

MARTIN, GEORGE EARL
Entered service Oct. 1, 1917; Capt., Engrs., N. A., Aug. 11, 1917. Overseas service Mar. 30, 1918–June 8, 1919. Discharged July 2, 1919. Supply Officer and Adj., 3d Bn., 23d Engrs.

MARTIN, JAMES WALTER
Capt., Engrs., U. S. A.*

MARTIN, LEWIS M.
Entered service Dec. 28, 1917, as Capt., Engrs., N. A. Discharged Apr. 2, 1918. Disqualified for active service by accident while in training.

MARX, RAYMOND
Entered service May 12, 1917; 2nd Lt., Inf., U. S. A., Aug. 15, 1917; 1st Lt., Inf., U. S. A., Sept. 15, 1918. Overseas service Apr. 4, 1918–May 10, 1919. Discharged May 28, 1919.

MASLEN, HAROLD CARPENDALE
Entered service Nov. 2, 1917, as Machinist's Mate, 2d Class, U. S. N.; Ensign, Feb. 2, 1918. Released from active service Jan. 31, 1919. Engr. in chg., insp. outside contracts, U. S. Navy Yard, Charleston, S. C.; eng. duties, U. S. S. *Wisconsin*. One star.

MASSEI, CAESAR
Entered service Mar. 29, 1917; Capt., C. A. C., U. S. A., Apr. 15, 1917; Maj., C. A. C., U. S. A., July 12, 1917; Maj., Engrs., U. S. A., Apr. 27, 1918. Overseas service June 30, 1918–Jan. 24, 1919. Discharged May 27, 1919. Battery Comdr., Ft. Monroe; Ft. Comdr., Fishermans Island; Bn. Comdr., 22d Engrs., constr. railway artillery; Instr. in Artillery; Camp Engr., at Fort Monroe and in France. Two stars.

MASSEY, GEORGE BRAGG
Entered service June 4, 1917, as Lt., Jr. Grade, U. S. N.; Lt., U. S. N., Feb. 1, 1918. Overseas service Dec. 19, 1917–Jan. 28, 1919. Released from active service Mar. 11, 1919. Chf. Engr., U. S. S. *Isla de Luzon;* U. S. Mine Force, Base No. 18, Inverness, Scotland.

MASSEY, WALTER F.
Entered service July 22, 1918, as Pvt., Engrs., N. A.; Sgt., 1st Class, Engrs., U. S. A., Dec. 28, 1918. Overseas service Oct. 18, 1918–July 9, 1919. Discharged July 17, 1919. With 129th Engrs.

MASTERS, FRANK MILTON
Entered service Nov. 21, 1917, as Maj., Ord. Dept., N. A. Discharged May 31, 1919. Personnel work, Washington, D. C.; Asst. Insp. Mgr., and Insp. Mgr., Phila. Ord. Dist.; Member, Claims Bd., Phila. Dist. Special citation from Maj. Gen. C. C. Williams, Chf. of Ord.

MATLAW, ISAAC SOLON
Entered service May 8, 1917; Capt., E. O. R. C., Jan. 26, 1917. Resigned Dec. 28, 1917. Co. Comdr., 23d Engrs., Acting Regt. Comdr.

MATTHEWS, WARREN SHEPARD
1st Class Pvt., Tank Corps, U. S. A.*

MAUL, THEODORE RUSSELL
Entered service Apr. 18, 1917; Capt., Q. M. R. C., June 3, 1916; Maj., Q. M. C., U. S. A., Feb. 13, 1919. Overseas service May 7, 1918–June 27, 1919. Asst. to Depot Q. M., Phila., in chg. constr. and repairs, etc.; Bn. Comdr., Stevedore Troops; Executive Officer, Depot Q. M., Gievres, France; Post. Q. M., Vals-les-Bains, France; Depot Officer, Pittsburgh, Pa.

MAURICE, GEORGE HOLBROOKE
Entered service May 8, 1917; Capt., E. O. R. C., June 13, 1917. Discharged, acct. physical disability, Dec. 11, 1917; 2d Lt., Q. M. C., U. S. A., Sept. 10, 1918. Discharged, Dec. 6, 1918. With 352d Labor Bn.

MAURY, DABNEY HERNDON
Entered service Jan. 7, 1918, as Maj., E. R. C., Constr. Div.; Lt. Col., Q. M. C., Constr. Div., U. S. A., Mar. 18, 1918. Discharged May 31, 1919. Advisory Engr. on water supply on all army projects U. S. or island possessions; special assignment to U. S. Marine Corps, at Paris Island and Quantico, and to Navy Dept. at Pearl Harbor, Bishop's Point, and Puuloa Reservation, and to War Industries Board in Hampton Roads Dist. of the Army, Navy, Shipping Board and Housing Corp.

MAXSON, FRANK OSCAR
Entered service Oct. 26, 1881; through all grades in C. E. C., U. S. N., to Capt., Nov. 26, 1906. Recalled to active service Oct. 6, 1917. Released from active service Nov. 3, 1919. Public Works Officer, 7th Naval Dist., U. S. Naval Station, Key West, Fla.

MAZEAU, CAMILLE
Entered service Apr. 2, 1917; 1st Lt., C. A. C., U. S. A., June 16, 1916; Capt., C. A. C., U. S. A., Mar. 26, 1918. Overseas service Mar. 28, 1918–Jan. 21, 1919. Discharged Jan. 25, 1919. With the 56th Artillery, in command Battery D; with French troops, and with 3d and 5th Army Corps, 1st Army, A. E. F.; operations at Nancy, Chateau-Thierry, Vesle River and the Argonne. Two stars.

MEANS, JOHN SIEMON
Entered service May 15, 1917; 2d Lt., E. O. R. C., Aug. 15, 1917; 1st Lt., Engrs., N. A., July 18, 1918; Capt., Engrs., U. S. A., Apr. 7, 1919. Overseas service Jan. 23, 1918–July 28, 1919. Discharged Aug. 17, 1919. Supply Officer and Adj. with 508th Engrs.; Asst. Adj. and Aide to Chf. Engr., A. E. F. Chevalier du Merite Agricole.

MEARS, FREDERICK
Entered service Feb. 1, 1918; Col., Engrs., U. S. A., Jan. 16, 1918. Overseas service June 6, 1918–May 22, 1919. C. O., 31st Engrs.; Asst. Gen. Mgr. and Gen. Mgr., Railroad Dept., S. O. S., Tours, France. Distinguished Service Medal; Officier, Legion d'Honneur.

MECHLIN, OSCAR ALEXANDER
Entered service July 5, 1917, as Lt., U. S. N.; Lt. Comdr., U. S. N., Aug. 29, 1918; Comdr., U. S. N., June 26, 1919. Public Works Officer, Phila. Navy Yard and 4th Naval Dist.

MELIN, REYNOLD FERDINAND
Entered service Aug. 27, 1917; 1st Lt., Ord. Dept., U. S. A., Nov. 27, 1917; Capt., Ord. Dept., U. S. A., Oct. 1, 1918. Experimental Engr. on ry. artillery, Washington, D. C., and Proving Grounds.

MELLON, ALBERT EMERSON
Entered service Sept. 1, 1917, as Capt., C. A. C., U. S. A. Overseas service Dec. 27, 1917–Jan. 12, 1919. Discharged Jan. 31, 1919. C. O., 2d Anti-Aircraft Battery, 1st Army, A. E. F.

MELTON, ARTHUR POMEROY
Entered service May 8, 1917, as Capt., E. O. R. C.; Maj., Engrs., U. S. A., Apr. 19, 1919. Overseas service Aug. 5, 1917–Nov. 17, 1918. Discharged Dec. 20, 1918. 1st Asst., Water Supply and Sewerage Sec., Lines of Communication; S. O. S., Water Supply and Sewerage Officer, Base Sec. No. 5, A. E. F.

MENDENHALL, HERBERT DRUMMOND
Entered service May 16, 1917, as Capt., E. O. R. C. Overseas service July 23, 1917–July 21, 1919. Discharged Aug. 13, 1919. Office, Chf. of Engrs., Washington, D. C.; in chg. hosp. constr., Beau Dersert and Talence, also Le Courneau Camp, Base Sec. No. 2; in chg. demountable barrack contracts and settling engineer contracts. Chevalier de l'Etoile Noire.

MENEFEE, FERDINAND NORTHRUP
Entered service Oct. 6, 1917, as Capt., Ord. Dept., U. S. A.; Maj., Ord. Dept., U. S. A., July 25, 1918. Discharged May 29, 1919. Chf., Engrs. of Tests, Metallurgical Branch, Insp. Div.; Metallurgist, Cleveland Dist.

MENGEL, CARL WAYNE
Entered service May 7, 1917; 2d Lt., E. O. R. C., Apr. 16, 1917; 1st Lt., E. O. R. C., Aug. 15, 1917; Capt., Engrs., N. A., Mar. 23, 1918. Overseas service July 30, 1918–June 15, 1919. Discharged June 21, 1919. Adj., 2d Bn., 306th Engrs. Two stars.

MERCKEL, FREDERICK GEORGE
Entered service July 16, 1918, as 2d Lt., San. C., Eng. Div., U. S. A.; 1st Lt., San. C., Eng. Div., U. S. A., May 3, 1919. Overseas service Sept. 29, 1918–Aug. 17, 1919. Discharged Sept. 3, 1919. Water Supply Service; Div. of Constr. and Forestry; Office, Chf. Engr., 3d Army, as Chf. of Water Purification Section.

MERRIAM, CHARLES ALLEN
Entered service June, 1917, as Capt., E. O. R. C. Discharged Jan. 23, 1919. Prin. Asst. to Const. Q. M., Tullytown Bag Loading Plant.

MERSHON, RALPH DAVENPORT
Entered service Jan. 23, 1917, as Maj., E. O. R. C.; Lt.-Col., Engrs., U. S. A., Oct. 30, 1918. Discharged Jan. 23, 1919. In chg. experimental work of Special Problems Committee, Naval Consulting Board.

MESSER, RICHARD
Entered service June 17, 1918, as Maj., Constr. Div., U. S. A. Discharged Mar. 14, 1919. In chg., Water and Sewer Sec., superv. operation and maintenance of water works and sewerage systems, etc., at army posts, camps and reservations.

MILES, GEORGE FREDERICK
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service Feb. 27, 1918–July 4, 1919. Discharged July 8, 1919. Adj., 8th Bn., 20th Engrs., Orleans Forestry Dist., and Le Mans, Baugé and Bourges Dists., France.

MILHAN, DAVID NELSON
Entered service May 11, 1917; 1st Lt., Engrs., N. A., Sept. 6, 1917. Overseas service Sept. 16, 1918–July 6, 1919. Discharged Aug. 1, 1919. With 106th Engrs. at Brest, France, on constr. Camp Pontanezen.

MILKOWSKI, VICTOR JOHN
Entered service May 8, 1917, as 2d Lt., E. O. R. C. Overseas service Dec. 14, 1917–June 25, 1918. Discharged Nov. 30, 1918. On detached service with 302d, 101st and 117th Engrs., A. E. F.

MILLER, CHARLES HENRY
Entered service July 19, 1917; Maj., Engrs., N. A., Dec. 10, 1917. Overseas service Mar. 30, 1918–Apr. 2, 1919. Discharged Apr. 5, 1919. Const. Q. M., Camp Cody; Bn. Comdr., 23d Engrs.; Dist. Engr. at Langres, France.

MILLER, DANIEL CHAMBERS
Entered service June 19, 1917, as 1st Lt., E. O. R. C. Overseas service June 28, 1918–July 8, 1919. Discharged July 17, 1919. With 516th Engrs.

MILLER, GEORGE SOTER
Entered service July 15, 1917, as Maj., Engrs., N. A. Overseas service Feb. 15, 1918–July 15, 1919. Dischargerd July 31, 1919. Assigned to Constr. Div., A. S., on constr. of Flying Fields and Air Depots; Chf., Metal and Machinery Div., A. S., A. E. F.; Member, Gen. Purchasing Bd., and Liquidation Comm., A. E. F. Two stars.

MILLER, HAROLD BROWN
1st Lt., Engrs., U. S. A.*

MILLER, HUGH
Capt., Engrs., U. S. A.*

MILLER, JOHN OWEN
Entered service Sept. 22, 1917, as 1st Sgt., Machine Gun Co., Inf., N. A.; 2d Lt., Inf., N. A., June 1, 1918; 2d Lt., Engrs., N. A., July, 1918. Discharged Feb. 24, 1920. Instr. at Camp Humphreys; Personnel Adj.

MILLER, KARL ANDREW
Entered service May, 1918, as Pvt., Engrs., N. A.; Sgt., Engrs., N. A., Aug. 1, 1918; Sgt., First Class, N. A., July 25, 1919. Overseas service July 10, 1918–Aug. 20, 1919. Discharged Aug. 29, 1919. With 29th Engrs., in chg. computation of triangulation and traverse, 3d Army. Citation from Gen. Pershing for meritorious service. One star.

MILLIS, JOHN
Entered service June, 1877; through all grades in C. of E., U. S. A., to Col., June 13, 1910. Div. Engr., Southeast Div.; Dist. Engr., Savannah Dist.; Dept. Engr., Southeast Dept., stationed at Savannah, Ga., throughout the war.

MILLS, ADELBERT PHILO
Entered service May, 1918, as Capt., Engrs, U. S. A. Overseas service 1918. Camp Lee and Fort Oglethorpe. Died Oct. 20, 1918, in service at Brest, France.

MILLS, GUY G.
Entered service June 28, 1917, as 1st Lt., E. O. R. C.; Capt., Engr. R. C., Feb. 25, 1918. Discharged May 16, 1919. Purchasing Officer, Constr. Div.; Adj., Mechanical Schools, Camp Humphreys.

MINER, ERWIN JOHN
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service July 9, 1918–Sept. 17, 1918. Discharged Mar. 24, 1919. Asst. to Div. Engr. Officer, 88th Div., Camp Dodge; Co. Comdr., 528th Engrs. Service Bn. in U. S. and overseas.

MINNISS, GEORGE STEWART
Entered service Aug. 5, 1917, as Maj., Inf., U. S. A. Discharged Oct. 31, 1919. Bn. Comdr., 55th Pioneer Inf.; Q. M., 2d Brig.; Bn. Comdr., 6th Bn. and 422d Reserve Labor Bn.; Pres., Maritime Investigation Board.

MITCHELL, CHARLES HAMILTON
Brig. Gen., Gen. Staff, B. E. F.* Distinguished Service Order; Companion of the Bath; Companion of the Order of St. Michael and St. George; Croix de Guerre; Ordre de Leopold, Belgium; Croce de Guerra and Officer of the Crown, Italy.

MOLITOR, FREDERIC A.
Entered service Feb., 1917, as Maj., E. O. R. C.; Lt. Col., Eng. R. C., Dec., 1917; Col., Engrs., U. S. A., Sept., 1918. Overseas service June 28, 1918–Mar. 9, 1919. Discharged Mar., 1919. Office Chf. of Engrs., Washington, D. C.; with 22d Engrs., Div. of Eng. Supplies. Citation from Gen. Pershing; Officier, Ordre de l'Etoile Noire.

MOLLER, IRVING CLARK
Entered service May 7, 1917; Capt., E. O. R. C., Jan. 26, 1917; Maj., E. O. R. C., Aug. 14, 1917. Overseas service Mar. 26, 1918–Feb. 9, 1919. Discharged Feb. 11, 1919. Bn. Comdr., 23d Engrs., in district of Perigeux, Dordogne, France; on staff, Brig. Gen. W. D. Connor; Depot Engr. Officer, Base Sec. No. 7, A. E. F.

MONCRIEFF, JOHN MITCHELL
Lt. Col., Royal Engrs. Director of Eng. Work, Admiralty (Controllers and C. G. M. S. Dept., 1917–18). Commander of the Order of the British Empire.

MONK, PERCY SHELLEY
Entered service Apr. 28, 1918; 1st Lt., Engrs., N. A., June 10, 1918. Overseas service Aug. 14, 1918–Mar. 10, 1919. Discharged Mar. 20, 1919. With 29th Engrs.; detached service, Sound Ranging School, Ft. de St. Menge, France; Sound Ranging Sec. No. 4. 2d Army, in Woevre Sector, A. E. F. One star.

MONROE, ROBERT ANSLEY
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., July 26, 1917; Capt., C. of E., U. S. A., July 12, 1918. Overseas service Apr. 30, 1918–Aug. 1, 1919. Resigned Sept. 20, 1919. With 4th Engrs. as Regtl. Supply Officer; Asst., G–1, 4th Div. Five stars.

MONTGOMERY, ALBERTIS
Entered service Sept. 2, 1917, as 1st Lt., Engr. R. C.; Capt., T. C., U. S. A., Feb. 14, 1919; 1st Lt., C. of E., U. S. A., July 1, 1920. Overseas service June 7, 1918–Aug. 17, 1919. With 31st Engrs.; North Russian Exp. Force; with 9th Engrs.

MONTGOMERY, JULIAN
Entered service May 10, 1918, as Pvt., 1st Class; 2d Lt., Inf., U. S. A., Aug. 26, 1918; 2d Lt., Engrs., U. S. A., Oct. 6, 1918. Discharged Dec. 3, 1918. With 162d Depot Brig., Camp Pike; E. O. T. S., Camp Humphreys.

MOORE, JAMES ARCHIBALD
Entered service July 20, 1917; 1st Lt., Engrs., N. A., Aug. 3, 1917; Capt., Engrs., N. A., Aug. 1, 1918. Overseas service Mar. 30, 1918–Apr. 13, 1919. Discharged May 19, 1919. Co. Comdr.; Bn. Adj., Regtl. Supply Officer, 110th Engrs., service at Vosges, St. Mihiel, Verdun, and Meuse-Argonne offensives. Two stars.

MOORE, LEWIS EUGENE
Entered service May 7, 1917; Capt., E. O. R. C., Apr. 16, 1917; Maj., Engr. R. C., June 14, 1918. Overseas service Oct. 27, 1917–Oct. 1, 1918. Discharged Dec. 23, 1918. In chg. field constr. and maintenance of highway bridges, 1st Army; design standard highway bridges, A. E. F.; in chg. bridge and road reconnaissance at the front for G. H. Q.; in chg. of obtaining special steel bridge equipment in U. S. for A. E. F.

MORE, CHARLES CHURCH
Entered service Sept. 2, 1917; Capt., Engr. R. C., June 19, 1917; Capt., Ord. R. C., Oct. 18, 1917; Maj. Ord. Dept., U. S. A., July 25, 1918. Discharged Oct. 1, 1919. With Constr. Sec., Supply Div., Ord. Dept., Washington, D. C.; Instr., Eng. School, Camp Humphreys.

MORELOCK, JOHN EARL
Entered service May 20, 1917; 2d Lt., E. O. R. C., June 27, 1917. Overseas service Dec. 11, 1917–Apr. 25, 1919. Discharged Apr. 30, 1919. With 306th Engrs., Camp Jackson; detached duty with Engr. Purchasing Office, A. E. F.; Bn. Adj., 508th Engrs.; with 112th Engrs. on special duty, Meuse-Argonne offensive. One star.

MOREY, EDWARD FRANCIS
Entered service Aug. 24, 1917, as Pvt., F. A., U. S. A.; Capt., F. A., U. S. A., Nov. 24, 1917. Overseas service May 22, 1918–Aug. 7, 1919. Discharged Aug. 19, 1920. With 2d Bn., Camp Stanley and A. E. F. Five stars.

MORGAN, THOMAS CHARLES
Entered service Apr. 5, 1918, as Pvt., F. A., U. S. A.; Cpl., F. A., U. S. A., July 1918; Sgt., F. A., U. S. A., Oct. 1, 1918. Overseas service Apr. 18, 1918–Aug. 8, 1919. Discharged Aug. 8, 1918. Student at University College, London; with 304th F. A. Citation from Gen. Alexander in General Orders for bravery under fire. Four stars.

MORIARTY, CLARENCE
Entered service Sept. 13, 1917, as 1st Lt., Engrs., N. A.; Capt., Engrs., U. S. A., Feb. 13, 1919. Overseas service July 5, 1918–May 22, 1919. Discharged May 28, 1919. Co. Comdr., 445th Truck Co.; C. O., Motor Reception Park, Base Sec. No. 6, A. E. F. Order of University Palms, Officier d'Academie, France.

MORRIS, CHARLES CHESTER
Entered service Oct. 21, 1918; 1st Lt., Engrs., N. A., Oct. 19, 1918. Discharged Nov. 30, 1918. Camp Humphreys.

MORRISON, ROGER LEROY
Capt., Engrs., U. S. A.*

MORRISON, WILLIAM GROVER
Entered service June 24, 1918, as Pvt., Engrs., N. A.; Cpl., Engrs., N. A., July, 1918; 2d Lt., Engrs., U. S. A., Nov. 7, 1918. Discharged Jan. 1, 1919. C. O., Ponton School Detachment, Camp Humphreys.

MORROW, BEN STOGDEN
Entered service Dec. 28, 1917; Capt., Engrs., N. A., May 1, 1918. Overseas service, June 1918–July, 1919. Discharged July 12, 1919. C. O., 529th Engrs.; Water and Sewer, Engr., Base Sec. No. 5, A. E. F., in chg. water supply for Navy at Brest, Camp Pontanezen, etc.

MORROW, DAVID CAMPSEY
Entered service Aug. 16, 1918; Capt., Q. M. C., Constr. Div., U. S. A., Aug. 9, 1918. Discharged July 25, 1919. Asst. to Utilities Officer, Camps Meade and Pike; in chg. constr. of sewage disposal plants, sewers, water supply and garbage disposal.

MORROW, JAY JOHNSON
Entered service June 15, 1887; through all grades in C. of E. to Brig. Gen., U. S. A., June 26, 1918. Overseas service Apr. 30, 1918–Dec. 30, 1918. C. O., 4th Engrs., and Chf.

Engr., 4th Div. and 1st Army; Deputy Chf. Engr., A. E. F. Officier, Legion d'Honneur. Four stars.

MORROW, SAMUEL ROY
Entered service May 1, 1918; 1st Lt., E. O. R. C., June 28, 1917. Overseas service July 10, 1918–Mar. 21, 1919. Discharged Apr. 7, 1919. With 29th and 74th Engrs., St. Mihiel and Meuse-Argonne Sectors. One star.

MORTON, LEON LINCOLN
Entered service Sept. 2, 1917, as Capt., Engrs., N. A.; Maj., Engrs., U. S. A., Sept. 18, 1918; Lt. Col., Engrs., U. S. A., Nov. 2, 1918. Overseas service Mar. 16, 1918–Apr. 12, 1919. Discharged May 2, 1919. Co. Comdr. and Bn. Comdr., 7th Engrs., 5th Div. Citation for gallantry in action. Three stars.

MOSS, CASTLE PRENTICE
Entered service Feb. 8, 1917; Sapper, Canadian Ry. Troops, Feb. 19, 1917; Cpl., Canadian Ry. Troops, Apr. 23, 1917. Overseas service Mar. 4, 1917–Mar. 1, 1919. Discharged Apr. 30, 1919. With 6th Ry. Troops on light ry. and standard gauge constr.; detached service with British Anti-Aircraft Battery.

MOSS, WILLIAM BENJAMIN
Entered service Jan. 3, 1918; Capt., E. O. R. C., May 5, 1917. Overseas service July 9, 1918–Mar. 24, 1919. Discharged June 20, 1919. Co. Comdr., 528th Engrs., Camp Dodge and A. E. F., hosp. constr., light railway work near Toul, 1st and 2d Armies.

MOWER, HARRISON CURTIS
Entered service May 14, 1917; Capt., E. O. R. C., June 9, 1917; Maj., Engrs., N. A., Aug. 15, 1917; Lt. Col., Engrs., N. A., May 2, 1918. Overseas service May 19, 1918–May 10, 1919. Discharged May 21, 1919. With 307th Engrs.; Toul Sector, Marbache Sector, St. Mihiel and Meuse-Argonne offensives. Three stars.

MUCKLESTON, HUGH BURRITT
Entered service Apr. 1, 1916, as Capt., 4th Pioneer Bn., C. E. F.; Capt., 1st Bn., Canadian Ry. Troops, Mar. 1, 1917; Maj., 1st Bn., Canadian Ry. Troops, Nov. 1, 1917. Overseas service Sept. 13, 1916–Mar. 2, 1919. Discharged Apr. 1, 1919. Mentioned in dispatches by Sir Douglas Haig.

MUENSTER, ROLAND AUGUST
Entered service Apr. 18, 1918, as Pvt., Sig. C., U. S. A.; Sgt., Sig. C., U. S. A., Nov. 1, 1918. Discharged Feb. 26, 1919. In chg. Meteorological Sta., Love Field, Dallas, Tex.; Science and Research Dept., Aviation Sec.

MUIRHEAD, JAMES HERBERT HAWKSWORTH
Entered service Aug., 1915, as Lt., Royal Engrs., British Army. Overseas service May 2, 1916–July 12, 1918. Released from active duty Nov. 1, 1919. Insp. Officer in chg. New York Dist., British Munitions Comm.; with Inland Waterways and Docks Div., Royal Engrs., in England, France and Italy; Lecturer in New York City, British Bureau of Information, and Deputy Asst. Provost Marshal.

MUNN, HARVEY TIMLOW
Entered service May 15, 1917; 1st Lt., E. O. R. C., June 16, 1917. Discharged June 17, 1918. Student, E. O. T. C.; Office, Chf. of Engrs., Washington, D. C., in chg., Water Supply Sec., under Officer in chg., Equipment Div.; with 26th Engrs., Camp Dix.

MUNOZ, GONZALO CLAUDIO
Entered service Sept., 1917, as Capt., Ord. Dept., U. S. A.; Maj., Ord. Dept., U. S. A., Dec., 1917. Discharged June, 1919. In chg., Constr. Sec., Supply Div., planning Ord. Storage Depots in U. S.

MURDOCK, ROBERT BRUCE
Entered service Feb. 28, 1918, as 2d Lt., Engrs., N. A.; 1st Lt., Engrs., N. A., Apr. 16, 1918; Capt., Engrs., U. S. A., Nov. 11, 1918. Overseas service May 15, 1918–May 29, 1919. Discharged June 9, 1919. With 42d Engrs. as Supply Officer; Post Exchange Officer, Camp American Univ.; Engr. Supply Officer, Savenay Hosp. Center, France; Base Engr. Adj., Base Sec. No. 1, A. E. F.

MURPHY, ALVIN RUSH
Entered service Jan. 3, 1918, as Capt., Engrs., N. A. Overseas service Jan. 15, 1918–Aug. 13, 1919. Discharged Sept. 9, 1919. Office of Chf. Engr., Water Supply Sec., Div. of Constr. and Forestry, A. E. F., on design and operation of water plants.

MURPHY, FRED ELMER
Capt., Ord., U. S. A.*

MURPHY, JAMES JOSEPH
Capt., Engrs., U. S. A., A. E. F.*

MURRAY, EVERETT BODMAN
Entered service Jan. 23, 1917; Capt., C. of E., U. S. A., Aug. 1, 1917; Maj., C. of E., U. S. A., July 30, 1918. Overseas service June 1, 1918–Oct. 1, 1918. With 314th Engrs.; assigned to Q. M. Constr. Div., organizing large water supply projects and Ord. Dept., Claims Bd., Gillespie Explosion.

MURRAY, JAMES POWELL
Entered service June 27, 1918, as Capt., Engrs., U. S. A. E. O. T. C., Camps Lee and Humphreys; Office of Chf. of Engrs., U. S. A., Washington, D. C. Died Sept. 28, 1918, in active service at Austin, Tex.

MUSHAM, JOHN WILLIAM
Entered service Sept. 2, 1917; Capt., Engr. R. C., July 26, 1917. Overseas service Apr. 29, 1918–June 22, 1919. Discharged July 17, 1919. With 108th Engrs.; Co. Comdr., 513th Engrs., Div. of Constr. and Forestry, A. E. F., on constr. of railroads, buildings and roads, Intermediate Sec., S. O. S. One star.

MYERS, EDMUND TROWBRIDGE DANA, JR.
Entered service Dec. 8, 1917; Maj., Ord. Dept., U. S. A., Aug., 1917. Discharged Mar. 11, 1919. In command New York Office, Nitrate Div.

NAGLER, FLOYD AUGUST
Entered service Apr. 19, 1918, as Pvt., Sig. C., U. S. A.; 2d Lt., Sig. C., U. S. A., Sept. 15, 1918. Discharged Jan. 24, 1919. Instr., Army School of Meteorology; Meteorological Officer, Mineola, L. I.; Insp., Meteorological Stas. in U. S.

NANCE, ARCHIBALD WHITFIELD
Entered service Apr. 10, 1918, as Pvt., Ord. Dept., N. A.; 2d Lt., Ord. Dept., N. A., July 18, 1918. Discharged Jan. 24, 1919. Ord. Supply School, Camp Hancock; Chf. of Guards and Supt. of Village, U. S. Nitrate Plant No. 1, Sheffield, Ala.

NASH, FRANKLYN DANA
Entered service July 21, 1917, as 1st Lt., Engrs., N. A.; 1st Lt., R. T. C., N. A., Feb. 5, 1918. Overseas service July 28, 1917–Jan. 13, 1919. Discharged Jan. 19, 1919. With 12th Engrs. on light ry. constr., Somme Battle Front, A. E. F.; laying out ry. yards, etc. at La Rochelle, Nantes, Marseilles, Paris. Two stars.

NEEDHAM, EGBERT STEPHEN
Entered service Apr. 9, 1918, as Pvt., Engrs., N. A.; Cpl., Engrs., N. A., June 1, 1918. Overseas service May 22, 1918–May 28, 1919. Discharged May 28, 1919. With 43d Engrs. and 46th Co., 20th Engrs.

NEEL, ARTHUR WOOD
Capt., R. T. C., U. S. A., A. E. F.*

NEFF, HENRY CONRAD
2d Lt., Engrs., U. S. A.*

NEINKEN, MORTIMER L.
Entered service June 28, 1917, as 2d Lt., E. O. R. C.; 1st Lt., Engrs., U. S. A., Aug. 8, 1918; Capt., A. S. C., U. S. A., Feb. 15, 1919. Overseas service Jan. 7, 1917–May 22, 1919. Discharged May 23, 1919. With 103d Engrs.; with Gas Service, A. E. F.; with A. S. C. Labor Bureau, A. E. F. One star.

NELSON, CLARENCE LOTARIO
Entered service July 16, 1917; Capt., E. O. R. C., July 24, 1917; Major C. A. C., U. S. A., Oct. 26, 1918. Overseas service Aug. 15, 1917–Apr. 20, 1919. Discharged Apr. 23, 1919. With 7th C. A. C. as Orientation Officer; Brig. Engr. Officer, Ry. Arty.; on staff, Ry. Arty. superv. orientation work of ry. arty. Citation from C. G., Ry. Arty. Reserve, A. E. F. Four stars.

NEUMAN, DAVID LEONARD
Entered service May 8, 1917, as 2d Lt., E. O. R. C.; Capt., Engrs., U. S. A., June 4, 1917; Maj., Engrs., U. S. A., Oct., 1918; returned to permanent rank of Capt., C. of E., U. S. A., Apr., 1919. Overseas service Dec., 1917–June, 1918. With 2d and 214th Engrs.; Executive Officer, Camp Forrest; in Canal Zone; Bn. Comdr., 3d Engrs. Two stars.

NEVILLE, COLONE WILL JACKSON
Entered service Mar. 28, 1917, as Lt. Comdr., U. S. N. R. F. Released from active service July, 1919. C. O., 3d Sec., 8th Naval Dist.

NEWHALL, WILLIAM BARRETT
Entered service Dec. 28, 1917, as Capt., Engrs., N. A. Discharged Aug. 27, 1919. Assigned to Constr. Div.; Chemical Plant No. 4; in Supervisor's Office, Washington, D. C.; on purchase of land, Camp Dodge.

NEWTON, JEWETT BEACH
Entered service May 14, 1918; 2d Lt., F. A. R. C., June 20, 1917; Capt., F. A., N. A., Aug. 25, 1917. Overseas service July 16, 1918–Jan. 20, 1919. Discharged Feb. 18, 1919. C. O. 301st Trench Mortar Battery.

NEWTON, JOHN PARSONS
Entered service Oct. 1, 1917; Capt., Engrs., N. A., Aug. 15, 1917. Overseas service Jan. 7, 1918–June 30, 1919. Discharged July 18, 1919. Deputy Sec. Engr., Advance Sec., A. E. F.; in charge constr. Base Hosp. No. 66; Convalescent Camp No. 2 and Motor Transport Park No. 1; Water Supply Officer for Langres Dist., France.

NIAL, WILLIAM AUGUSTINE
Entered service Dec. 28, 1917; Capt., Engrs., N. A., Sept. 26, 1917. Overseas service Oct. 27, 1918–June 6, 1919. Discharged June 30, 1919. Personnel Officer, Office Chf. of Engrs., Washinton, D. C.; Instr. Training Camps, Camps Lee and Humphreys; Co. Comdr., 540th Engrs.

NICHOLS, ARTHUR CLOUGH
Entered service April 8, 1918, as Pvt., Engrs., N. A.; 2d Lt., Engrs., U. S. A., Oct. 30, 1918 Discharged Jan. 15, 1919. Constr. standard gauge rys. to Camp Humphreys; with 45th and 210th Engrs.

NICHOLS, CHARLES HENRY
Entered service Aug. 8, 1917; Maj., E. O. R. C., June 13, 1917. Discharged Sept. 11, 1919. Assigned to Const. Q. M., Camp Kearny; Superv. Const. Q. M., Washington, D. C., in chg hosp. constr.; Officer in chg. constr., Camp McArthur; Officer in chg. constr., U. S. Gen. Hosp. No. 11; Acquisition Officer, Camp Sherman.

NILES, ALFRED SALEM, JR.
Entered service May 28, 1918; 2d Lt., Engrs., U. S. A., May 15, 1918; Capt., Engrs., U. S. A., Aug. 20, 1918; returned to provisional 1st Lt. Engrs., U. S. A., Apr. 1, 1919. Discharged Nov. 9, 1919. With 7th Engr. Training Regt., Camp Humphreys; with 79th Engrs. Camp Leach; with 3d Engrs. in Canal Zone.

NIMMO, JAMES VALENCE
Entered service Sept., 1914, as Lt., Royal Engrs., British Army; Capt., Royal Engrs., Ry. Constr. Troops, Aug., 1915. Overseas service Feb. 15, 1915–Apr. 15, 1919. Discharged Aug., 1919. With 111th Ry. Co. and Ry. Constr. Engrs., Lines of Communication; with 273d Ry. Co., light ry. work in Palestine; Educational Officer, France and England.

NIXON, COURTLAND
Entered service July 9, 1898, as 2d Lt., Inf., U. S. A. Retired in 1915 but returned to active duty May 15, 1917, as Maj., Q. M. C.; Lt. Col., Q. M. C., U. S. A., Aug. 19, 1918; Col., Q. M. C., U. S. A., July 19, 1919. Overseas service June 1918–Jan. 1919. Returned to retired list, Mar. 31, 1920. Asst. to Depot Q. M., New York City; in chg. of Material Control Office for Clothing Production, New York City; Div. Q. M., 83d Div.; Graduate, Gen. Staff College, Langres, France, and assigned to G-3, 2d Army. One star.

NOBLE, GUY LYNN
Entered service Feb. 13, 1918, as Capt., Engrs., N. A. Discharged Nov. 29, 1918. Asst. to Const. Q. M. and Executive Officer, Newport News, Va.; Officers Training School, Camp Humphreys.

NOLAN, SIMON FRANK
Entered service July 25, 1917; Maj., C. A. C., N. A., Aug. 5, 1917. Overseas service July 30, 1918–June 30, 1919. Discharged July 21, 1919. C. O., Ft. Standish and Ft. Warren, Boston Harbor; Bn. Comdr., 71st Arty., C. A. C.; in charge of sec. to determine damage to rys., roads and canals in Italy for Peace Commission; in command troops guarding U. S. material in transit through France, Germany and Holland. One star.

NOLAN, THOMAS BREW, JR.
Capt., Engrs., U. S. A.*

NOLAND, CUTHBERT POWELL
Capt., Engrs., U. S. A., A. E. F.*

NORDWELL, ALFRED WORCESTER
Entered service Oct. 14, 1918, as Capt., Engrs., U. S. A. Discharged Feb. 1, 1919. With 403d Engrs., Ft. Douglas; E. O. T. S., Camp Humphreys.

NORRIS, CLAIRE GREEN
Entered service Sept. 30, 1918, as 1st Lt., Engrs., U. S. A. Discharged Feb. 19, 1919. E. O. T. S., Camp Humphreys; with 55th Engrs., Ft. Leavenworth; with 219 Engrs., Camp Dodge.

NORTH, ROBERT GASTON
Entered service May 3, 1918; Capt., Engrs., N. A., Jan. 28, 1918. Overseas service Oct. 1, 1918–June 18, 1919. Discharged June 29, 1919. Topographic Officer, Intelligence Officer and Prison Officer, 605th Engrs.

NOSKA, GEORGE ALBERT
Entered service May 15, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service May 27, 1918–June 6, 1919. Discharged July 2, 1919. Supply Officer, 303d Engrs. and 303d Engr. Train; C. O., 303d Engr. Train; Regtl. Transport Officer; St. Mihiel, Meuse-Argonne and Limey Sector operations. Three stars.

NOWLIN, ROBERT ALDRIDGE
Entered service May 15, 1918; 2d Lt., Engrs., U. S. A., Nov. 6, 1918. Discharged Jan. 17, 1920. Asst. Personnel Adj., Camp Humphreys.

NOYES, ARTHUR PAGE
Entered service Dec. 4, 1917; Capt., M. T. C., U. S. A., July 25, 1917. Overseas service Jan. 5, 1918–Oct. 5, 1918. Discharged Dec. 31, 1918. In chg. Motor Reception Park, Havre, France; gen. convoy duty.

NOYES, STEPHEN HENLEY
Entered service Feb. 5, 1917, as Sgt., A. S. R. C.; 1st Lt., A. S. R. C. May 10, 1917; Capt., A. S., N. A., Aug. 1, 1918; Maj. A. S., U. S. A., Apr. 23, 1919. Overseas service Aug. 13, 1917–May 19, 1919. Discharged May 27, 1919. Flight Comdr., 1st Aero Squadron; C. O., 12th Aero Squadron, 5th Corps Observation Group, and Corps Observation Group, 1st Army; Chateau-Thierry, St. Mihiel and Meuse-Argonne offensives. Distinguished Service Cross; Croix de Guerre. Three stars.

OAKES, JOHN CALVIN
Entered service June 15, 1893; Lt. Col., C. of E., U. S. A., May 15, 1917; Col. Engrs., N. A., Aug. 15, 1917; Col., C. of E., U. S. A., July 1, 1920. Overseas service July 31, 1918–May 5, 1919. In chg. Philadelphia Engr. Dist.; C. O., 113th Engrs.; C. O., 5th Engrs. and Div. Engr., 7th Div.; Corps Engr., 6th Corps; Dist. Engr., Norfolk, Va. One star.

OBER, RALPH HADLOCK
Entered service Aug. 17, 1918, as Capt., Engrs., U. S. A. Discharged May 24, 1920. With 2d Engr. Training Regt., Camp Humphreys; with 5th Engrs., Camp Humphreys; Camp H. Q Detachment and Engr. School, Camp Humphreys.

O'CONNELL, GEORGE PAUL
Capt., Engrs., U. S. A.*

ODONI, VINCENT PHILLIP
Entered service June 21, 1918, as Ensign, C. E. C., U. S. N.; Lt., Jr. Grade, C. E. C., U. S. N., Apr. 1, 1919. Released from active service Sept. 10, 1920. Public Works Office, Navy Yard, San Diego, Cal., in chg. constr. Marine Corps Base as Project Mgr.

OGDEN, MERTON MILES
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., June 25, 1917. Overseas service May 8, 1918–June 2, 1919. Discharged July 8, 1919. With 33d Engrs.

OKES, DAY IRA
Entered service Dec. 28, 1917; Capt., E. O. R. C., Oct. 18, 1917; Maj., Engrs., N. A., July 31, 1918. Overseas service May 10, 1918–Sept. 1, 1918. Discharged Nov. 29, 1918. Capt. and Adj., 42d Engrs., American Univ. and in France; in chg. highway school, Camp Humphreys; returned from overseas to train engr. troops.

OLBERG, CHARLES REAL
Capt., Engrs., U. S. A., A. E. F.*

O'MEARA, ROBERT JOSEPH
1st Lt., Engrs., U. S. A., A. E. F.*

OPDYCKE, HENRY GORTON
Maj., Sig. C., U. S. A.*

OPENSHAW, JOHN EDWARD
Entered service Apr. 15, 1918, as Lt., Canadian Engrs. Overseas service July 10, 1918–Jan. 12, 1919. Released from active service Feb. 10, 1919. With Canadian Engrs. at Camp Seaford, England.

ORBECK, MARTIN J.
Entered service Dec. 28, 1917; 1st Lt., Engr., R. C., Oct. 3, 1917; Capt., Engrs., U. S. A., Oct. 18, 1918. Overseas service July 9, 1918–July 18, 1919. Discharged Aug. 6, 1919. Bn. and Personnel Adj., 528th Engrs., Advance Sec., S. O. S., A. E. F.; instr. and in chg. Academic Dept., Educational Center, Lerouville, France. Citation from Gen. Pershing for meritorious service. Two stars.

O'ROURKE, BERNARD JOHN
Entered service Nov., 1917; 1st Lt., A. S., N. A., Jan., 1918. Overseas service Mar. 14, 1918–Dec. 1, 1918. Discharged Jan. 7, 1919. C. O., 5th Constr. Co., Air Service.

ORT, ALBERT AUGUST LAMBERT
Entered service Mar. 30, 1918, as Ensign, C. E. C., U. S. N. R. F.; Lt., Jr. Grade, C. E. C., U. S. N. R. F., Dec., 1918; Lt., C. E. C., U. S. N. R. F., June 23, 1919. Asst. to Public Works Officer, 4th Naval Dist.; Trans. Officer, Philadelphia Navy Yard; Officer in chg. contract work, Public Works Dept., Philadelphia Navy Yard.

OSBORN, IRWIN SELDEN
Entered service Jan. 22, 1918, as Maj., Q. M. C., N. A. Discharged Jan. 24, 1919. Chf. of Reclamation Div. (later Salvage Div.), Office of Q. M. Gen.

OSBORNE, GEORGE FREDERICK FOLGER
Maj., Royal Engrs., B. E. F.*

OSTROM, CHARLES DOUGLAS YELVERTON
Entered service Nov. 28, 1916, as 2d Lt., C. A. C., U. S. A.; 1st Lt., C. A. C., U. S. A., Nov. 28, 1916; Capt. C. A. C., N. A., Aug. 5, 1917; Maj., C. A. C., U. S. A., Oct. 26, 1918. Overseas service July 14, 1918–Dec. 21, 1919. Returned to permanent rank of Capt., C. A. C., U. S. A., Mar. 17, 1920. Co. Comdr. and Adj., Coast Defenses of Pensacola; school at Ft. Monroe; with 64th Arty. as Bn. Adj. and Bn. Comdr.; Deputy Director of Military Affairs and Business Mgr., T. C., A. E. F.; with 31st Arty. Brig.

OSTRUP, JOHN CHRISTIAN †
Entered service May 26, 1917; Maj., Engrs., U. S. A., Jan. 23, 1917. Overseas service Oct. 5, 1917–Apr. 25, 1918. Discharged Jan. 11, 1919. On duty at Plattsburg Barracks and Camp American University; superv. constr. of trenches, fortifications, and bridges with A. E. F.; Bn. Comdr., 1st Bn., 4th Engrs.; Instr., Machine Gun Training Center, Camp Hancock; with engr. replacement troops, Camp Forrest.

OUTWATER, HERBERT GREGOR
Entered service Sept. 26, 1917; 1st Lt. San. C., N. A., Oct. 4, 1917; Capt., San. C., N. A., Jan. 20, 1918. Discharged Dec. 20, 1918. Camp Engr. Officer, Camp Greenleaf and Camp Oglethorpe; C. O., San. Sec. No. 77, 38th Div., Camp Shelby.

OWEN, ELIJAH HUNTER
Entered service Aug. 23, 1918, as Capt., Engrs., U. S. A. Discharged May 19, 1919. With 143d Engrs. at Camp Shelby; various duties with 2d Engr. Training Regt., Camp Humphreys; Regtl. J. A., 5th Engrs.

OXER, GEORGE CARROL
Entered service May 29, 1918; Maj., Engrs., N. A., May 23, 1918. Overseas service June 9, 1918–Aug. 20, 1919. Discharged Sept. 11, 1919. In chg. constr., St. Sulpice Storage Depot and St. Loubes Ammunition Depot; Asst. Sec. Engr., Base Sec. No. 2; attached American Commission to Negotiate Peace, Paris, in chg. Gen. Bldg. Sec., appraisal of damages; Dist. Road Engr. Indre-et-Loire, France, and Post Engr., H. Q., S. O. S., A. E. F.

PAGE, EDWIN RANDOLPH
Entered service Jan. 1, 1918, as 1st Lt., A. S., N. A. Overseas service Aug. 16, 1918–Feb. 15, 1919. Technical Officer, Mechanics Training School, Wendover, England; special duties H. Q., A. S., London.

† Died Feb. 27, 1919.

PAGON, WILLIAM WATTERS
Entered service Feb. 14, 1918, as Capt., Q. M. C., N. A. Discharged Mar. 15, 1919. 1st Asst. to Const. Q. M., Camp Meade and Curtis Bay Ord. Depot; acting Const. Q. M., Curtis Bay Ord. Depot.

PAINTER, PENNELL CHURCHMAN
Entered service Sept. 10, 1917, as 1st Lt., Engrs., N. A.; Capt., Engrs., U. S. A., Sept. 28, 1918. Overseas service July 10, 1918–July 1, 1919. Discharged July 31, 1919. Co. Comdr., 2d Engr. Training Regt., Camp Humphreys; Co. Comdr., 524th Engrs. St. Mihiel and Meuse-Argonne offensives. Two stars.

PALMER, WALLACE CROMWELL ALLEN
Entered service June 17, 1918, as Capt., Engrs., N. A. Discharged Dec. 5, 1918. E. O. T. S., Camp Humphreys.

PARKE, SAMUEL REYNOLDS
2d Lt., San. C., U. S. A.*

PARKER, HENRY BRACKETTE
Entered service Dec. 11, 1917; Q. M., 2d Class, U. S. N., Feb. 1, 1918; Q. M., 1st Class, U. S. N., May 1, 1918; Chf. Carpenter's Mate, U. S. N., Aug. 1, 1918. Overseas service Feb. 17, 1918–Dec. 15, 1918. Discharged Jan. 24, 1919. Asst. to Public Works Officer, Paimboeuf, France.

PARKER, JAMES
Entered service Nov. 10, 1917, as 2d Lt., Cavalry, N. A.; 1st Lt. Cavalry, N. A., Nov., 1917. Discharged Jan. 1, 1919. With 13th Cavalry on Mexican Border, Brownsville Dist.

PARKER, JAMES LAFAYETTE
Entered service Sept. 13, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 4, 1918. E. O. T. S., Camp Humphreys.

PARKER, WILLIAM EDWARD
Entered service Sept. 24, 1917, as Lt. Comdr., U. S. N. R. F. Released from active service, Mar. 20, 1919. Design, purchase and installation and supervision of compasses on naval vessels.

PARKS, CHARLES WELLMAN
Entered service July 19, 1897, as Lt., Jr. Grade, C. E. C., U. S. N.; Rear-Admiral, C. E. C., U. S. N., Jan. 12, 1918. Chf., Bureau of Yards and Docks, Navy Dept.; Ch. of C. E. C., U. S. N. Distinguished Service Medal.

PARSONS, ARCHIBALD LIVINGSTONE
Entered service Mar., 1903, as Lt., Jr. Grade, C. E. C., U. S. N.; Comdr., C. E. C., U. S. N., July, 1916. Asst. Chf., Bureau of Yards and Docks, Navy Dept.; Public Works Officer, Philadelphia Navy Yard; Chf. Engr., Republic of Haiti. Navy Cross.

PARSONS, CHARLES EDWARD
Entered service Aug. 27, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 24, 1918. Camp Humphreys.

PARSONS, HAROLD ASHTON
Entered service Nov. 4, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Mar. 1, 1919. Asst. Const. Q. M., Perryville, Md.; stationed at Washington, D. C.; Ord. Claims Board, Gillespie explosion, Perth Amboy, N. J.

PARSONS, WILLIAM BARCLAY
Entered service Oct. 16, 1916, as Maj., E. O. R. C.; Lt. Col., E. O. R. C., July 7, 1917; Col., Engrs., U. S. A., Sept. 27, 1918. Overseas service May 14, 1917-Apr. 27, 1919. Discharged May 22, 1919. With 11th Engrs.; Sr. Member, Engr. Commission; on staff Chf. Engr., A. E. F. Distinguished Service Medal; Chevalier, Order of the Crown Belgium. Citation from Gen. Pershing.

PARTRIDGE, JOHN FREDERICK
Entered service Sept. 2, 1917; 1st Lt., Engrs., N. A., Nov. 1, 1917; Capt., Engrs., U. S. A., Oct. 28, 1918. Overseas service Mar. 30, 1918–June 9, 1919. Discharged July 9, 1919. With 23d Engrs. in road constr. and repair work.

PATRICK, MASON MATHEWS
Entered service Sept. 1, 1882; Brig. Gen., N. A., Aug. 5, 1917; Maj. Gen., N. A., June 26, 1918. Overseas service Aug. 7, 1917–July 20, 1919. C. O., 1st Engrs.; in chg. all engr. training, A. E. F.; Chf. Engr., Lines of Communication, A. E. F.; Director, Constr. and Forestry, A. E. F.; Chf. of Air Service, A. E. F. Permanent rank, Col., C. of E., U. S. A. Distinguished Service Medal; Commandeur, Legion d'Honneur; Commander, Order of St. Maurice and Lazarus, Italy.

PATSTONE, LEWIS FREDERICK
Entered service June 19, 1918, as Maj., Engrs., N. A. Discharged Feb. 4, 1919. C. O., Camp No. 3, Camp Shelby; Bn. Comdr., 217th Engrs., Camp Beauregard.

PATTERSON, CHARLES SCOTT
Entered service Sept. 25, 1917; 2d Lt., Engr., R. C., Aug. 23, 1917; Capt. C. W. S., U. S. A., Nov. 13, 1918. Overseas service Jan. 23, 1918–Jan. 24, 1919. Discharged Feb. 12, 1919. With 113th Engrs.; with 1st Engrs.; Asst. Div. Gas Officer, 61st Div., 78th Div. and 28th Div., Montdidier and Toul Sectors; St. Mihiel and Meuse-Argonne offensives. Torpedoed on S. S. *Tuscania*. Four stars.

PATTERSON, LAURENCE
Entered service Mar., 1917, as Cpl., Engr. R. C.; Sgt., Engrs. N. A., July, 1917; Sgt., 1st Class, Engrs., N. A., Mar., 1918; Pvt., Q. M. C., N. A., May 6, 1918; 1st Lt., Q. M. C.,

Constr. Div., U. S. A., Aug. 20, 1918. Discharged Mar. 24, 1919. Designed and supervised constr. sewer system, Camp Wadsworth.

PATTON, WILLIAM BAIRD
Maj., Engrs., U. S. A.*

PAYNE, EDWIN VAN RENSSELAER
Entered service May 8, 1917; Maj., Engrs., N. A., July 12, 1917; Lt. Col., Engrs., N. A., Jan. 17, 1918. Overseas service Feb. 26, 1918–May 14, 1919. Discharged June 18, 1919. Sec. Engr. Base Sec. No. 5, A. E. F.; C. O., 25th Engrs. in Meuse-Argonne offensive; with 1st Army in chg. Gen. Constr. Sec. Citation from Gen. Pershing; Chevalier, Legion d'Honneur. Two stars.

PAYNE, LOUIS WATTERS
Entered service May 24, 1918, as Pvt., Engrs., N. A.; M. E., Sr. Grade, Engrs., N. A., July 1, 1918; 2d Lt., Engrs., U. S. A., Apr. 18, 1919. Overseas service July 31 1918–July 5, 1919. Discharged July 25, 1919. With 22d Engrs. on light ry. constr. in Toul Sector; 1st Army. Two stars.

PEARCE, RUFUS BURLESON
Entered service May 15, 1917; 2d Lt., E. O. R. C., June 19, 1917; 1st Lt., Engrs., N. A., Aug. 15, 1917; Capt., Engrs., N. A., June 13, 1918. Overseas service July 21, 1918–June 27, 1919. Discharged July 21, 1919. With 310th and 314th Engrs.; served with 5th Corps, A. E. F., and 7th Corps, 3d Army. Two stars.

PEASE, HAROLD TAPLEY
Entered service May 11, 1917; Capt., F. A., N. A., Sept., 1917. Overseas service June 27, 1918–Apr. 20, 1919. Discharged Dec. 24, 1919. C. O., 316th Trench Mortar Battery; Bn. Comdr., 12th Provisional Regt., Trench Arty.

PECK, JOHN SANFORD
Entered service Mar. 1, 1918, as Pvt., A. S., N. A.; Sgt., A. S. A., May 15, 1918; 2d Lt., A. S. A., Aug. 19, 1918. Discharged Jan. 6, 1919. With Photo Sec., Kelly Field; School of Aerial Photography, Ithaca, N. Y.

PECK, MYRON HALL
Entered service 1917 as Capt., E. O. R. C. Overseas service, 1918. Instr., E. O. T. C., Camp Lee; Co. Comdr. and Bn. Comdr., 2d Engrs. Chateau Thierry, St Mihiel Drive. Killed Oct. 9, 1918, in action near St. Etiennes-a-Armes, France. Posthumous honor of the Distinguished Service Medal.

PENNELL, JAMES ROY
Entered service July 25, 1917, as Capt., Engrs., N. A. Overseas service Oct. 14, 1917–Jan. 19, 1919. Discharged Jan. 21, 1919. Co. Comdr., 117th Engrs.; with Ry. Arty. Three stars.

PENSE, EDWARD HERBERT
Lt., Canadian Engrs., B. E. F.*

PERKINS, ALVA HAROLD
Entered service July 1, 1917; Capt., E. O. R. C., June 23, 1917. Overseas service July 28, 1917–May 30, 1919. Discharged Aug. 20, 1919. Co. Comdr., 17th Engrs.; Staff Engr. Officer, 51st Arty., C. A. C. Croix de Guerre. Three stars.

PERKINS, ROSCOE, JR.
Entered service May 25, 1918, as Pvt., F. A., N. A.; 2d Lt., F. A., U. S. A., Oct., 1918. Released from active duty Jan. 6, 1919. With 311th F. A.; with 4h Regt. F. A. Replacement Depot, as Instr. in topography and use of instruments.

PERKINS, SETH, JR.
Entered service Aug. 27, 1917; 2d Lt., F. A., N. A., Nov. 27, 1917. Overseas service July 13, 1918–May 4, 1919. Discharged May 27, 1919. With 11th and 120th F. A.; Meuse-Argonne offensive. One star.

PERRIN, LESTER WILLIAM
Entered service May 15, 1917; Capt., Inf., N. A., Aug. 15, 1917. Overseas service July 3, 1918–July 6, 1919. Discharged July 10, 1919. Co. Comdr., 301st Inf.; detached service at St. Nazaire and at Peace Conference.

PERRINE, GEORGE
Entered service Apr. 27, 1918, as Maj., Q. M. C., Constr. Div., U. S. A. Discharged Oct. 25, 1919. Asst. Const. Q. M., Army Supply Base, Brooklyn, N. Y.

PERRY, ARTHUR FRANKLIN, JR.
Entered service Aug. 27, 1917; 2d Lt., C. A. C., N. A., Nov. 27, 1917. Overseas service July 13, 1918–Feb. 24, 1919. Discharged Apr. 9, 1919. Orientation Officer, 64th Arty., C. A. C.

PERRY, CHARLES EDWARDS
Entered service Aug. 25, 1917; Capt., Engrs., N. A., Oct. 1, 1917; Maj., Engrs., U. S. A., Aug. 15, 1918. With 213th Engrs.

PERRY, FRANCIS WILLIAM
Entered service May 15, 1917; Capt., E. O. R. C., Apr. 16, 1917; Maj., Engrs., N. A., July 30, 1918. Overseas service, March, 1918–Oct., 1918. Discharged Dec. 5, 1918. Co. Comdr. 302d Engrs.; Mobilization Officer, Q. M., and Deputy Chf. of Staff, 77th Div.; Gen. Staff College, A. E. F.; Gen. Staff, 1st Army, as Secy. Operations Sec. and Chf. Sub-Sec. Operations. With 77th Div. in Hazebrouck area and Baccarat Sector. Four stars.

PERRY, LYNN ELWOOD
Entered service May 15, 1917; 1st Lt., E. O. R. C., Apr. 18, 1917; 1st Lt., San. C., N. A., Mar. 6, 1918; Capt., San. C., U. S. A., Jan. 27, 1919. Overseas service July 17, 1918–Aug. 12, 1919. Discharged Sept. 2, 1919. San. Insp., Taylor Field, Ala.; with San. Squadron No. 43, 27th Div.; San. Insp., Mars-sur-Allier and Kerhoun. Two stars.

PETERS, ALBERT AYER
Entered service May 15, 1917; Capt., E. O. R. C., May 5, 1917; Maj., Engrs., N. A., Mar. 7, 1918; Lt. Col., C. W. S., U. S. A., Feb. 14, 1919. Overseas service Mar. 30, 1918–July 28, 1919. Discharged Aug. 12, 1919. Co. Comdr., Regtl. Adj., and Bn. Comdr., 23d Engrs.; transferred to C. W. S., Aug., 1918, and established special training schools in A. E. F.; C. O., Gen. Training Area and activities; Personnel Officer, C. W. S., A. E. F. Cited in Gen. Orders by Chf. of C. W. S. Officier, French Academy. One star.

PHALAN, JOHN JOSEPH FRANCIS
Entered service Dec. 28, 1917; 1st Lt., Engrs., N. A., Oct. 1, 1917. Discharged Apr., 1918. Camp Lee.

PHILIPS, HECTOR SOMERVILLE
Entered service Feb. 21, 1917, as Lt., Canadian Engrs. Discharged Jan. 9, 1919. On staff of Engr. Training Depot, St. Johns, Que.

PHILIPS, JAMES HARRY
Entered service May 2, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged Apr. 16, 1919. Asst. to Sec. Chf., Sec. C, Terminals and Depots; Advisory Engr. on roads and railroads; in chg. maintenance, roads and railroads.

PHILLIPS, WALTER BELLEVILLE
Entered service May 1, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Sept. 19, 1919. Asst. Const. Q. M., Jefferson Barracks Base Hosp.; Asst. Const. Q. M. and Officer in Chg. Eng. Properties Coast Defenses of New York; Asst. Const. Q. M., Ord. Storage Depot, Sandy Hook; Surveying Officer, Army Supply Base, Brooklyn, N. Y.

PHIPPS, THOMAS ELMER
Entered service Dec. 28, 1917; Capt., E. O. R. C., June 19, 1917. Discharged Feb. 5, 1919. Engr. Officer in Chag. Constr., New York Depot.

PICKETT, FRANK HURD
Capt., Engrs., U. S. A.*

PIERCE, CHARLES HENRY
Entered service Dec. 28, 1917; 1st Lt., E. O. R. C., Feb. 14, 1917. Discharged Apr. 20, 1918. E. O. T. S., Camp Lee.

PIERCE, CHARLES WILLIAM
Entered service Apr., 1917, as Ensign, E. D. O., U. S. N. R. F.; Lt., Jr. Grade, E. D. O., U. S. N. R. F., July 1, 1918; Lt., E. D. O., U. S. N. R. F., Jan. 1, 1919. Overseas service Apr., 1917–Mar., 1919. Discharged Mar., 1919. Engr. Officer with Pacific Fleet in South America and with Atlantic Fleet on East Coast of U. S.

PIERCE, PAUL LEON
Entered service June 1, 1917 as Maj., Ord. R. C.; Lt. Col., Ord. C., N. A., Jan. 13, 1918. Overseas service Feb., 1918–Feb., 1919. Discharged Mar. 1, 1919. Executive Office Inspection Div., Ord. Dept.; Chf. of Purchase Div., Office Chf. Purchasing Officer, A. E. F. Chevalier, Legion d'Honneur; Certificate of meritorious service.

PIERCE-HOPE, JOHN
Entered service Aug. 10, 1917; Feb. 16, 1918, with Red Cross Sec., French Army; Conducteur. Feb. 27, 1918. Overseas service Aug. 15, 1917-July 31, 1918. Discharged July 31, 1918. Motor Ambulance Service on northwest front, Amiens-Rheims Sec. Shellshocked.

PILL, LEON MORLEY
Entered service May 15, 1917; Capt., Engrs., N. A., Aug. 14, 1917; Maj., Engrs., U. S. A., Aug. 31, 1918. Overseas service Nov. 12, 1917–July 5, 1919. Discharged July 28, 1919. With 308th and 20th Engrs.; Gen. Staff, G. H. Q., A. E. F. Citation from Gen. Pershing

PILLSBURY, GEORGE BIGELOW
Entered service June 15, 1896; Maj., C. of E., U. S. A., at declaration of war; Col., Engrs., N. A., Aug. 15, 1917. Overseas service Aug. 7, 1918–May 30, 1919. C. O., 115th Engrs., C. O., 102d Engrs.; Corps Engr., 2d Corps.; Office, Chf. Engr., A. E. F.

PIODA, ALBERT WOODBRIDGE
Entered service Oct., 1918; Capt., Engrs., U. S. A., Oct. 23, 1918. Trans. Officer, and Camp Adj., Camp Humphreys.

PIRNIE, HERBERT MALCOLM
Entered service March 7, 1918, as 1st Lt., T. C., N. A.; Capt., T. C., U. S. A., Oct. 27, 1918. Overseas service Mar. 28, 1918–Mar. 21, 1919. Discharged Mar. 26, 1919. Asst. Engr. of Water Supply, H. Q.; Director Gen. of Transportation, A. E. F.

PLANK, DAVID HARLAN
Entered service Sept. 25, 1917; 1st Lt., Engrs., N. A., Apr. 3, 1918. Overseas service July 6, 1918–Aug. 16, 1919. Discharged Aug. 25, 1919. Adj., 516th Engrs. and Engr. Sub-Post, Gen. Intermediate Storage Depot, Gievres, France; Officer in chg. rock supply, Paris and Intermediate Dist. Road Sec.

PLUMER, HAROLD EDWARD
Entered service Oct. 3, 1918, as Maj., Q. M. C., Constr. Div., U. S. A. Discharged Dec. 31, 1918.

POND, FREDERICK HENRY
Entered service Apr. 27, 1918; 1st Lt., E. O. R. C., Feb. 1, 1918; Capt., Engrs., U. S. A., Apr. 7, 1919. Overseas service Aug. 16, 1918–Aug. 6, 1919. Discharged Aug. 28, 1919. Co. Comdr., 34th Engrs., engr. depot work at St. Nazaire, Nantes, and elsewhere in France.

POOLE, CHARLES ARTHUR
Entered service Aug. 31, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 23, 1918. E. O. T. S., Camp Humphreys; with 150th Engrs., Camp Shelby.

POOLE, JOHN HUDSON
Entered service May 6, 1917; Maj., E. O. R. C., Nov. 15, 1916; Lt. Col., Engr. R. C., July 9, 1918; Col., Engrs., U. S. A., Sept. 19, 1918. Overseas service July 31, 1917–Jan. 21, 1919. Discharged Jan. 24, 1919. Bn. Comdr., 16th Engrs.; Gen. Staff, G. H. Q., A. E. F. Officier, Legion d'Honneur; Commandeur, Ordre de l'Etoile Noire.

POORMAN, ALFRED PETER
Entered service May 13, 1917; Capt., Engrs., N. A., July 11, 1917. Overseas service Oct. 31, 1917–June 28, 1919. Discharged July 17, 1919. Co. Comdr., 25th and 29th Engrs.; Supply Officer, G-2-C, G. H. Q., A. E. F.

PORTER, ELMER ALFRED
Entered service Oct. 16, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 18, 1918. With 403d Engrs.; E. O. T. S., Camp Humphreys.

PORTER, HENRY GEORGE
Capt., San. C., U. S. A.*

PORTER, RALPH WALDO
1st Lt., Engrs., U. S. A.*

POST, ADOLPH JOSEPH
Entered service Oct. 5, 1917, as Pvt., Inf., N. A.; Sgt., 1st Class, Q. M. C., N. A., Dec. 7, 1917; 2d Lt., Q. M. C., Constr. Div., N. A., May 16, 1918. Discharged Sept. 19, 1919. Officer in chg. of roads, eng. and boundary survey, Utilities Div., Camp Devens.

POST, WILLIAM SCHUYLER
Entered service May 24, 1917, as Capt., E. O. R. C.; Maj., Eng. R. C., Aug. 15, 1917. Overseas service July, 1918–June, 1919. Discharged July 15, 1919. Bn. Comdr., 316th Engrs.; on staff Chf. Engr. 1st Army, also 5th Army Corps; with Sec. Engr. Intermediate Sec. West, A. E. F. One star.

POTTER, CHARLES LEWIS
Col., Engrs., U. S. A.*

POWELL, ARCHIBALD OLIN
Entered service Aug. 2, 1917: Maj., E. O. R. C., Apr. 16, 1917; Lt. Col., Engrs., N. A., July 23, 1918. Military Asst. to Dist. Engr., 2d Dist.; Asst. Dept. Engr., Eastern Dept.; Enlisted Personnel Officer, Office, Chf. of Engrs., Washington, D. C.; in chg. Eng. Data Branch, Military Div., Office, Chf. of Engrs.

POWELL, HERBERT JAMES BINGHAM
Entered service Oct., 1915, as Staff Officer, British War Mission in U. S. In America from Nov., 1915, to Jan., 1919. Discharged Feb., 1919. Staff Officer, also executive of the British Govt. and American War Dept. in technical matters for the joint Dept. of Gauges and Standards of the Mission and U. S. Bureau of Aircraft Production. Officer of the Order of the British Empire.

POWELL, THOMAS JETT
Entered service May 12, 1917; Capt., E. O. R. C., Apr. 23, 1917; Maj., Engr. R. C., June 30, 1918. Overseas service May 9, 1918–Aug. 21, 1918. Topographic Officer and Engr. Supply Officer, 305th Engrs.

POWELL, WILLIAM JENNER
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 11, 1917; Maj., T. C., U. S. A., May 2, 1919. Overseas service June 30, 1918–June 30, 1919. Discharged July 16, 1919. Co. Comdr. and Bn. Adj., 66th Engrs.; Personnel Adj., 15th Grand Div., T. C.; Highway Div., Peace Commission; Transportation Guard Service, T. C.

POWELSON, WILFRID VAN NEST
Lt. Comdr., U. S. N.*

PRAEGER, EMIL
Entered service Dec. 27, 1917, as Lt., Jr. Grade, C. E. C., U. S. N.; Lt., C. E. C., U. S. N., July 1, 1918. Discharged June 11, 1920. Public Works Dept., 12th and 3d Naval Dists.

PRATT, ARTHUR HENRY
Entered service May 8, 1917; Capt., E. O. R. C., Jan. 22, 1917; Maj., Engrs., U. S. A., Oct. 31, 1918. Overseas service Oct. 30, 1917–Mar. 23, 1919. Discharged July 25, 1919. Student and Asst. Instr., Ft. Oglethorpe and American Univ.; Co. Comdr., 26th Engrs., front line water supply, and C. O., 26th Engrs.. C. O., 1st Bn., 26th Engrs.; Officer in Chg. Water Supply Sec., Office, Chf. Engr., 2d Army and Chf. Engr., Paris Group; Office, Chf. of Engrs., Troop Div. Training Sec. Diploma from Lt. Gen. Bullard for meritorious service. Two stars.

PRESTON, HARRY LONGYEAR
Entered service Aug. 10, 1918; Capt., Engrs., N. A., July 27, 1918. Overseas service Sept. 25, 1918–June 23, 1919. Discharged July 11, 1919. Co. Comdr. and Supply Officer, 22d Engrs.; Office Engr., Div. of Light Rys., Toul Sector; Meuse-Argonne offensive. One star.

PRICE, PHILIP WALLIS
Entered service May 31, 1917; 2d Lt., F. A., N. A., Aug. 15, 1917. Overseas service July 14, 1918–May 22, 1919. Discharged May 24, 1919. With 3d, 310th and 107th F. A.

PRICE, WILLIAM EDMUND
Entered service July 27, 1918, as Capt., Q. M. C., Const. Div., N. A. Discharged Dec. 23, 1918. Asst. to Const. Q. M., Baltimore Depot Warehouses and Camp Hancock.

PRICHETT, FREDERICK BORRADAILE
Entered service July 25, 1917, as Pvt., Engrs., N. A.; Col., Jan. 1, 1918; 2d Lt., F. A., N. A., Apr. 20, 1918. Overseas service, 1918. With 103d Engrs.; with 103d Trench Mortar Battery; O. T. C., Camp Hancock; C. O., H. Q. Co. and Battery Officer, 109th F. A., 28th Div. Died Sept. 6, 1918, from wounds received in action crossing Vesle River, France.

PRINDLE, FRANKLIN COGSWELL
Rear-Admiral, C. E. C., U. S. N. (*Retired.*)*

PRITCHARD, JOHN CHARLES
Entered service Aug. 15, 1917, as 1st Lt., Engrs., N. A.; Capt., Engrs., N. A., Feb. 14, 1918. Overseas service June 29, 1918–Mar. 12, 1919. Discharged Mar. 18, 1919. With 26th Engrs. serving with 3d Army Corps and 1st Army Corps as Dist. Water Supply Officer; installed water points for constr. Verdun-Conflans Ry.; with 3d Army Corps in front of Fismes; Toul Sector; St. Mihiel and Meuse-Argonne offensives. Two stars.

PROCTOR, RALPH FENNO
Entered service May 28, 1917; Maj., Q. M. C., Cantonment Div., June 19, 1917; Lt. Col., Q. M. C., Constr. Div., U. S. A., Aug. 28, 1918. Discharged July 15, 1919. Const. Q. M., Camp Meade and at Curtis Bay Ord. Depot.

PRUETT, GROVER CLEVELAND
Entered service Nov. 8, 1918, as Capt., Engrs., U. S. A. Discharged Mar. 25, 1919. Constr. work Ft. Keogh, Mont.

PUCKETT, HOMER
Entered service May 17, 1918; Capt., Engrs., N. A., May 7, 1918. Discharged Oct. 30, 1919. Member, Bd. of Officers on Heavy Ponton Equipage; head of bridge development unit, Office, Chf. of Engrs., Washington, D. C.

PUGH, MARSHALL ROGERS
Entered service June 13, 1917; Maj., E. O. R. C., Sept. 9, 1917. Overseas service Dec. 26, 1917–Dec. 29, 1918. Discharged Jan. 9, 1919. Bn. Comdr., 21st Engrs. in U. S. and at Nevers, France, in Toul Sector and St. Mihiel offensive; Post Engrs., Bordeaux Embarkation Camp. One star.

PULLIGNY, JEAN LECLERC de
Entered service Nov. 15, 1914, as Commandant, 1st Pioneers, French Army. Discharged June, 1917. In command of the 1st Group of roadbuilders, French 8th Army. Croix de Guerre.

PUNG, WILLIAM SING-CHONG
Entered service Oct. 3, 1917, as Pvt., Inf., N. A.; Cpl., Inf., N. A., Nov. 9, 1917; Sgt., Inf., N. A., Apr. 30, 1918; Sgt., Engrs., N. A., May 10, 1918; 2d Lt., Engrs., N. A., June 6, 1918; 1st Lt., Engrs., U. S. A., Nov. 4, 1918. Discharged Jan. 15, 1919. With 357th Inf.; Instr. in reconnaissance E. O. T. S., Camp Humphreys.

PURCELL, STEUART
Entered service May 12, 1917; Capt., E. O. R. C., June 23, 1917. Discharged Feb. 18, 1919. Engr. Depot, Hoboken; Member, Traveling Examining Bd. for Engr. Officers; Office, Chf. of Engrs.

QUICK, RAY STEVENS
Entered service Aug. 17, 1918; 1st Lt., Engrs., N. A., Aug. 6, 1918. Discharged Jan. 7, 1919. E. O. T. S., and 3d Engr. Training Regt.

QUILTY, T. FRANK
Entered service Feb. 28, 1918, as Maj., Engr. R. C. Discharged May 29, 1919. Constr. Div.; Asst. to Superv. Q. M., Washington, D. C.; in chg. and as Const. Q. M., completed Army Reserve Depot, Columbus, Ohio.

QUINBY, EDWIN RUFUS
Capt., Engrs., U. S. A., A. E. F.*

QUINLAN, GEORGE AUSTIN
Maj., Engrs., U. S. A., A. E. F.*

QUINN, JOHN IGNATIUS
Entered service May 8, 1917; 1st Lt., E. O. R. C., Apr. 5, 1917. Overseas service Nov. 26, 1917–Mar. 6, 1919. Discharged Mar. 26, 1919.

QUINN, MATTHEW FRANCIS
Maj., Engrs., U. S. A., A. E. F.*

QUINN, THOMAS FRANCIS
Entered service Sept. 25, 1917; 1st Lt., E. O. R. C., Aug. 8, 1917. Overseas service Jan. 22, 1918–May 31, 1919. Discharged July 2, 1919. In training Ft. Leavenworth; with 35th, 15th and 501st Engrs.

RADER, FRANKLIN KEARNS
Entered service Aug. 19, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 29, 1918. With 5th Engr. Training Regt., Camp Humphreys.

RADER, JAMES WILSON
Entered service Jan., 1918, as Pvt., 1st Class, A. S., N. A.; 2d Lt., A. S., Mar. 25, 1918. Discharged Jan., 1919. Instr., attached to Cadet Wing, H. Q. Staff, Kelly Flying Field.

RAKESTRAW, CHARLES LYSANDER
Entered service Sept. 2, 1917; 1st Lt., Engrs., N. A., Nov., 1917. Overseas service Apr. 29, 1918–Sept., 1918, and Jan. 15, 1919–July 7, 1919. Discharged Oct. 24, 1919. Bn. Gas Officer, 4th Engrs.; employed on destruction German ammunition dumps. Three stars.

RANDLE, GEORGE NELSON
Entered service Aug. 31, 1918, as Maj., Q. M. C., Constr. Div., U. S. A. Discharged Dec. 2, 1918. Utilities Officer, Camp Fremont and Camp Cody.

RANDOLPH, JOHN HAMPDEN, JR.
Capt., Engrs., U. S. A.*

RANDOLPH, ROBERT ISHAM
Entered service June 5, 1917, as Capt., E. O. R. C.; Maj., Engr. R. C., Dec. 17, 1917. Overseas service Aug. 5, 1918–Apr. 25, 1919. Discharged Apr. 29, 1919. With 23d Engrs.; organized, and C. O., 535th Engrs., attached to 2d Army; standard gauge ry. constr., St. Mihiel Sector. One star.

RANNEY, ALFRED GARDNER
Entered service July 27, 1917; Capt., C. A. R. C., Nov. 27, 1917. Overseas service Mar. 25, 1918–Jan. 30, 1919. Discharged Feb. 20, 1919. Bn. Comdr., 65th Arty.; St. Mihiel and Meuse-Argonne offensives. Two stars.

RATHJENS, GEORGE WILLIAM
Entered service Apr. 27, 1917; Capt., E. O. R. C., Feb. 23, 1917; Maj., E. O. R. C., Aug. 15, 1917; Lt. Col., Engr. R. C., Jan. 4, 1918. Overseas service Aug. 4, 1917–May 30, 1919. Discharged June 17, 1919. With 313th Engrs.; with 2d Army; staff, Chf. of Engrs. Three stars.

RAYMOND, HERBERT DWIGHT
Entered service Nov. 6, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 12, 1918. E. O. T. S., Camp Humphreys.

REAM, WARD HALL
Entered service 1917, as Capt., E. O. R. C.; on account of youth was rerated as 1st Lt., E. O. R. C. Recommendation for promotion to Capt. forwarded to Headquarters Oct. 2, 1918. Overseas service, 1918. O. T. S., Camp Myers; with 305th Engrs.; Co. Comdr., 305th Engrs. Killed Oct. 4, 1918, in action in France.

REDING, JAMES HANDYSIDE
Entered service July 14, 1917, as Capt., Engrs., N. G. (previous federal service from June, 1916, as Pvt., 2d Lt., and 1st Lt., Engrs., with Ohio N. G.). Overseas service June 23, 1918–Mar. 31, 1919. Discharged May 3, 1919. Student, 2d Corps School and Army School, France; Co. Comdr., 112th Engrs. in Baccarat Sector, Avacourt Sector, Meuse-Argonne offensive, Pannes Sector and Ypres-Lys offensive; Supply Officer, Serbian Mission, American Peace Commission, War Damages Bd. Three stars.

REED, CARL SWEETLAND
Entered service Oct. 14, 1918, as Maj., Ord. Dept., U. S. A.; Lt. Col., Ord. Dept., U. S. A., May 21, 1919; Col., Ord. Dept., U. S. A., Aug. 21, 1919. Discharged Oct. 31, 1919. Ord. Dist. Chf., New York Dist.

REED, HOWARD SAWYER
Entered service Dec. 28, 1917; Capt., Engrs., U. S. A., July 16, 1917. Discharged Oct. 30, 1919. Office Chf. of Engrs., Washington, D. C.; Gen. Staff, Portland, Ore.; Regtl. Adj., 216th Engrs.; Representative Adj. Gen. Dept., Camp Devens.

REED, PAUL LYON
Entered service Nov. 6, 1902, as Lt., Jr. Grade, C. E. C., U. S. N.; Comdr., C. E. C., U. S. N., July 1, 1917. Represented Emergency Fleet Corp., Hog Island constr.; Member Compensation Bd., Navy Dept.

REED, PERCY LAWRENCE
Entered service July 25, 1918, as 1st Lt., Ord. Dept., N. A.; Capt., Ord. Dept., U. S. A., May 29, 1919. Discharged Oct. 31, 1919. Duty at Nitrate Plant No. 2, Muscle Shoals, Ala.; Property Officer and C. O., Nitrate Plant No. 3, Toledo, Ohio.

REEDER, HARRY CALVIN
Entered service Jan. 3, 1918; 1st Lt., Engrs., N. A., Dec. 18, 1917. Discharged Apr. 3, 1918. E. O. T. C., Camp Lee.

REES, BIRD LeGRAND
Entered service Apr. 15, 1918; Capt., Engrs., N. A., Apr. 5, 1918. Discharged May 31, 1919. Constr. Div., Washington, D. C.; cantonment constr., Camp Humphreys; with 5th Engr., Training Regt., Camp Humphreys; Office, Chf. of Engrs., Washington, D. C.

REEVES, CARL HOWELL
Entered service July 19, 1917; Capt., E. O. R. C., Apr. 23, 1917; Maj., Q. M. C., Constr. Div., N. A., Mar. 18, 1918. Discharged June 4, 1919. Engr. Officer in chg. constr. Camp Fremont, Camp Doniphan, Ft. Sill and Camp Humphreys; Supervising Const. Q. M., Coast Defenses, Puget Sound; Const. Q. M., Camp Cody.

REEVES, GLENN STANTON
Entered service Dec. 15, 1917, as Pvt., Engrs., N. A.; Pvt., 1st Class, Engrs., N. A., May, 1918; Cpl., Engrs., U. S. A., Aug. 15, 1918. Overseas service Mar. 30 1918–July 21, 1919. Discharged July 26, 1919. With 23d Engrs. Two stars.

REICHARDT, WALTER FREDERICK
Entered service Aug. 9, 1917; Maj., Q. M. C., Constr. Div., May 12, 1917. Discharged Oct. 11, 1917. Asst. to Const. Q. M., Camp Beauregard; transferred to staff of 39th Inf.

REILLY, CHARLES GILBERT
Entered service May, 1917; 1st Lt., Inf., Aug. 1917; Capt., Inf., U. S. A., Aug., 1918. Overseas service, 1918. O. T. C., Fort Niagara; with 313th Inf., at Camp Meade and in France; Co. Comdr., 313th Inf. Killed Sept. 29, 1918, in action in Argonne Forest, France.

REIMER, ARTHUR ADAMS
Entered service May 8, 1917; Capt., E. O. R. C., Jan. 18, 1917; Maj., E. O. R. C., Aug. 15, 1917. Discharged May 6, 1918. Instr., Officers' Training Camp; with 108th and 305th Engrs.

REIMER, FREDERIC ADAMS
Entered service Apr. 24, 1917, as Capt., E. O. R. C.; Maj., E. O. R. C., May 5, 1917. Discharged Feb. 26, 1918. Organized 1st Bn., 104th Engrs.; topographical survey, Camp Dix.; gen. engr. work, Camp Lee; survey work and gen. camp eng. duty, Camp McClellan.

RENSHAW, ALFRED
Entered service May 11, 1917; Capt., E. O. R. C., Aug. 15, 1917. Overseas service, Mar. 29, 1918–Apr. 5, 1919. Discharged Apr. 12, 1919. With 302d Engrs. Three stars.

RENSHAW, FRANCIS OREA
1st Lt., Engrs., U. S. A.*

RESWICK, SOLOMON
1st Lt., Engrs., U. S. A.*

REYNOLDS, LEO FRANCIS
Entered service Dec. 8, 1917, as Pvt., 1st Class, Aviation Sec., Sig. R. C.; 2d Lt., A. S. A., N. A., June 5, 1918. Discharged Jan. 6, 1920. Ground school, Univ. of Texas; concentration camp, Camp Dick; Instr., Love Field; bombing plane training, Ellington Field.

REYNOLDS, RALPH WHITNEY
Entered service Oct. 1, 1917, as 2d Lt., Engr. R. C.; 1st Lt., Engr. R. C., Jan. 3, 1918. Overseas service, June 28, 1918–Mar. 20, 1919. Discharged Apr. 18, 1919. With 316th, 318th, and 37th Engrs. Aisne-Marne, St. Mihiel, and Meuse-Argonne offensives; Defense Sector. Four stars.

RHOADES, THEODORE ECKFORD
Entered service May 1, 1917; Capt., E. O. R. C., Feb. 22, 1917; Capt., G. S., A. E. F., July, 1918; Maj., G. S., U. S. A., Aug. 22, 1918. Overseas service Jan. 15, 1918–Feb. 1, 1919. Discharged Mar. 2, 1919. Const. Q. M. and Officer in chg. constr., Camp Sherman; Asst. Chf. of Staff, 83d Div.; student officer with 11th British Div.; attached to G–4, G. H. Q., and Paris Group, A. E. F.; Asst. Chf. of Staff, G–4, Advanced G. H. Q., Argonne and Germany. Four stars.

RHODES, ERIC HOUGHTON
Lt., Australian Field Engrs., B. E. F.*

RICE, JOHN TURNER
Entered service Dec. 28, 1917; Capt., E. O. R. C., Apr. 18, 1917; Maj., Engr. U. S. A., Feb. 13, 1919. Overseas service Sept. 20, 1918–July 23, 1919. Discharged July 30, 1919. With 2d Engr. Training Regt. and with 541st Engrs.

RICH, EDWARD DUNBAR
Entered service Aug. 12, 1918; Maj., San. C., U. S. A., Aug. 1, 1918. Discharged Dec. 27, 1918. Director, School of San. Eng., Camp Greenleaf.

RICH, MELVIN S.
Entered service May 12, 1917; 1st Lt., E. O. R. C., June 19, 1917. Overseas service Feb. 4, 1918–Feb. 2, 1919. Discharged Feb. 4, 1919. Cantonment constr.; Office, P. B. and G., Washington, D. C.; Labor Bureau, A. E. F.

RICHARDS, WALTER ALLAN
Entered service May 11, 1917; 2d Lt., Inf., N. A., Aug. 29, 1917; 1st Lt., Inf., N. A., Jan. 1, 1918. Overseas service Apr. 29, 1918–Mar. 16, 1919. Discharged Sept. 10, 1919. With 326th Inf. Distinguished Service Cross. Brigade citation for patrol work. Two stars. Two wounds.

RICHARDSON, FREDERICK HOSEA
Entered service Dec. 27, 1917; Capt., Engr. R. C., Sept. 19, 1917. Overseas service July 10, 1918–July 7, 1919. Discharged Aug. 1, 1919. Co. Comdr. and C. O., 526th Engrs.; in chg. constr., Div. Area No. 18 and No. 3, Advance Sec., A. E. F.; C. O., troops at Longuyon, France; in chg. bldg. constr., Univ. of Beaune, France.

RICHARDSON, JAMES HERBERT
Entered service Sept. 25, 1917; Capt., E. O. R. C., Aug. 15, 1917. Overseas service Jan. 20, 1918–June 2, 1918. Discharged Nov. 29, 1918. Co. Comdr., 306th Engrs., Camp Jackson; student, 2d Corps School, France; Co. Comdr., 5th Engr. Training Regt., Camp Humphreys; on duty, Office, Chf. of Engrs., Washington, D. C., revising engr. manual.

RICHARDSON, REX DENSMORE
Capt., Engrs., U. S. A.*

RICHE, CHARLES SWIFT
Entered service July 1, 1882; Col., C. of E., U. S. A., July 1, 1916. Dept. Engr., Central Dept. and Panama Canal Dept.; river and harbor and fortification work.

Constr. Div., U. S. A., Aug. 20, 1918. Discharged Mar. 24, 1919. Designed and supervised constr. sewer system, Camp Wadsworth.

PATTON, WILLIAM BAIRD
Maj., Engrs., U. S. A.*

PAYNE, EDWIN VAN RENSSELAER
Entered service May 8, 1917; Maj., Engrs., N. A., July 12, 1917; Lt. Col., Engrs., N. A., Jan. 17, 1918. Overseas service Feb. 26, 1918–May 14, 1919. Discharged June 18, 1919. Sec. Engr. Base Sec. No. 5, A. E. F.; C. O., 25th Engrs. in Meuse-Argonne offensive; with 1st Army in chg. Gen. Constr. Sec. Citation from Gen. Pershing; Chevalier, Legion d'Honneur. Two stars.

PAYNE, LOUIS WATTERS
Entered service May 24, 1918, as Pvt., Engrs., N. A.; M. E., Sr. Grade, Engrs., N. A., July 1, 1918; 2d Lt., Engrs., U. S. A., Apr. 18, 1919. Overseas service July 31 1918–July 5, 1919. Discharged July 25, 1919. With 22d Engrs. on light ry. constr. in Toul Sector; 1st Army. Two stars.

PEARCE, RUFUS BURLESON
Entered service May 15, 1917; 2d Lt., E. O. R. C., June 19, 1917; 1st Lt., Engrs., N. A., Aug. 15, 1917; Capt., Engrs., N. A., June 13, 1918. Overseas service July 21, 1918–June 27, 1919. Discharged July 21, 1919. With 310th and 314th Engrs.; served with 5th Corps, A. E. F., and 7th Corps, 3d Army. Two stars.

PEASE, HAROLD TAPLEY
Entered service May 11, 1917; Capt., F. A., N. A., Sept., 1917. Overseas service June 27, 1918–Apr. 20, 1919. Discharged Dec. 24, 1919. C. O., 316th Trench Mortar Battery; Bn. Comdr., 12th Provisional Regt., Trench Arty.

PECK, JOHN SANFORD
Entered service Mar. 1, 1918, as Pvt., A. S., N. A.; Sgt., A. S. A., May 15, 1918; 2d Lt., A. S. A., Aug. 19, 1918. Discharged Jan. 6, 1919. With Photo Sec., Kelly Field; School of Aerial Photography, Ithaca, N. Y.

PECK, MYRON HALL
Entered service 1917 as Capt., E. O. R. C. Overseas service, 1918. Instr., E. O. T. C., Camp Lee; Co. Comdr. and Bn. Comdr., 2d Engrs. Chateau Thierry, St Mihiel Drive. Killed Oct. 9, 1918, in action near St. Etiennes-a-Armes, France. Posthumous honor of the Distinguished Service Medal.

PENNELL, JAMES ROY
Entered service July 25, 1917, as Capt., Engrs., N. A. Overseas service Oct. 14, 1917–Jan. 19, 1919. Discharged Jan. 21, 1919. Co. Comdr., 117th Engrs.; with Ry. Arty. Three stars.

PENSE, EDWARD HERBERT
Lt., Canadian Engrs., B. E. F.*

PERKINS, ALVA HAROLD
Entered service July 1, 1917; Capt., E. O. R. C., June 23, 1917. Overseas service July 28, 1917–May 30, 1919. Discharged Aug. 20, 1919. Co. Comdr., 17th Engrs.; Staff Engr. Officer, 51st Arty., C. A. C. Croix de Guerre. Three stars.

PERKINS, ROSCOE, JR.
Entered service May 25, 1918, as Pvt., F. A., N. A.; 2d Lt., F. A., U. S. A., Oct., 1918. Released from active duty Jan. 6, 1919. With 311th F. A.; with 4h Regt. F. A. Replacement Depot, as Instr. in topography and use of instruments.

PERKINS, SETH, JR.
Entered service Aug. 27, 1917; 2d Lt., F. A., N. A., Nov. 27, 1917. Overseas service July 13, 1918–May 4, 1919. Discharged May 27, 1919. With 11th and 120th F. A.; Meuse-Argonne offensive. One star.

PERRIN, LESTER WILLIAM
Entered service May 15, 1917; Capt., Inf., N. A., Aug. 15, 1917. Overseas service July 3, 1918–July 6, 1919. Discharged July 10, 1919. Co. Comdr., 301st Inf.; detached service at St. Nazaire and at Peace Conference.

PERRINE, GEORGE
Entered service Apr. 27, 1918, as Maj., Q. M. C., Constr. Div., U. S. A. Discharged Oct. 25, 1919. Asst. Const. Q. M., Army Supply Base, Brooklyn, N. Y.

PERRY, ARTHUR FRANKLIN, JR.
Entered service Aug. 27, 1917; 2d Lt., C. A. C., N. A., Nov. 27, 1917. Overseas service July 13, 1918–Feb. 24, 1919. Discharged Apr. 9, 1919. Orientation Officer, 64th Arty., C. A. C.

PERRY, CHARLES EDWARDS
Entered service Aug. 25, 1917; Capt., Engrs., N. A., Oct. 1, 1917; Maj., Engrs., U. S. A., Aug. 15, 1918. With 213th Engrs.

PERRY, FRANCIS WILLIAM
Entered service May 15, 1917; Capt., E. O. R. C., Apr. 16, 1917; Maj., Engrs., N. A., July 30, 1918. Overseas service, March, 1918–Oct., 1918. Discharged Dec. 5, 1918. Co. Comdr. 302d Engrs.; Mobilization Officer, Q. M., and Deputy Chf. of Staff, 77th Div.; Gen. Staff College, A. E. F.; Gen. Staff, 1st Army, as Secy. Operations Sec. and Chf. Sub-Sec. Operations. With 77th Div. in Hazebrouck area and Baccarat Sector. Four stars.

PERRY, LYNN ELWOOD
Entered service May 15, 1917; 1st Lt., E. O. R. C., Apr. 18, 1917; 1st Lt., San. C., N. A., Mar. 6, 1918; Capt., San. C., U. S. A., Jan. 27, 1919. Overseas service July 17, 1918–Aug. 12, 1919. Discharged Sept. 2, 1919. San. Insp., Taylor Field, Ala.; with San. Squadron No. 43, 27th Div.; San. Insp., Mars-sur-Allier and Kerhoun. Two stars.

PETERS, ALBERT AYER
Entered service May 15, 1917; Capt., E. O. R. C., May 5, 1917; Maj., Engrs., N. A., Mar. 7, 1918; Lt. Col., C. W. S., U. S. A., Feb. 14, 1919. Overseas service Mar. 30, 1918–July 28, 1919. Discharged Aug. 12, 1919. Co. Comdr., Regtl. Adj., and Bn. Comdr., 23d Engrs.; transferred to C. W. S., Aug., 1918, and established special training schools in A. E. F.; C. O., Gen. Training Area and activities; Personnel Officer, C. W. S., A. E. F. Cited in Gen. Orders by Chf. of C. W. S. Officier, French Academy. One star.

PHALAN, JOHN JOSEPH FRANCIS
Entered service Dec. 28, 1917; 1st Lt., Engrs., N. A., Oct. 1, 1917. Discharged Apr., 1918. Camp Lee.

PHILIPS, HECTOR SOMERVILLE
Entered service Feb. 21, 1917, as Lt., Canadian Engrs. Discharged Jan. 9, 1919. On staff of Engr. Training Depot, St. Johns, Que.

PHILIPS, JAMES HARRY
Entered service May 2, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged Apr. 16, 1919. Asst. to Sec. Chf., Sec. C, Terminals and Depots; Advisory Engr. on roads and railroads; in chg. maintenance, roads and railroads.

PHILLIPS, WALTER BELLEVILLE
Entered service May 1, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Sept. 19, 1919. Asst. Const. Q. M., Jefferson Barracks Base Hosp.; Asst. Const. Q. M. and Officer in Chg. Eng. Properties Coast Defenses of New York; Asst. Const. Q. M., Ord. Storage Depot, Sandy Hook; Surveying Officer, Army Supply Base, Brooklyn, N. Y.

PHIPPS, THOMAS ELMER
Entered service Dec. 28, 1917; Capt., E. O. R. C., June 19, 1917. Discharged Feb. 5, 1919. Engr. Officer in Chag. Constr., New York Depot.

PICKETT, FRANK HURD
Capt., Engrs., U. S. A.*

PIERCE, CHARLES HENRY
Entered service Dec. 28, 1917; 1st Lt., E. O. R. C., Feb. 14, 1917. Discharged Apr. 20, 1918. E. O. T. S., Camp Lee.

PIERCE, CHARLES WILLIAM
Entered service Apr., 1917, as Ensign, E. D. O., U. S. N. R. F.; Lt., Jr. Grade, E. D. O., U. S. N. R. F., July 1, 1918; Lt., E. D. O., U. S. N. R. F., Jan. 1, 1919. Overseas service Apr., 1917–Mar., 1919. Discharged Mar., 1919. Engr. Officer with Pacific Fleet in South America and with Atlantic Fleet on East Coast of U. S.

PIERCE, PAUL LEON
Entered service June 1, 1917 as Maj., Ord. R. C.; Lt. Col., Ord. C., N. A., Jan. 13, 1918. Overseas service Feb., 1918–Feb., 1919. Discharged Mar. 1, 1919. Executive Office Inspection Div., Ord. Dept.; Chf. of Purchase Div., Office Chf. Purchasing Officer, A. E. F. Chevalier, Legion d'Honneur; Certificate of meritorious service.

PIERCE-HOPE, JOHN
Entered service Aug. 10, 1917; Feb. 16, 1918, with Red Cross Sec., French Army; Conducteur, Feb. 27, 1918. Overseas service Aug. 15, 1917-July 31, 1918. Discharged July 31, 1918. Motor Ambulance Service on northwest front, Amiens-Rheims Sec. Shellshocked.

PILL, LEON MORLEY
Entered service May 15, 1917; Capt., Engrs., N. A., Aug. 14, 1917; Maj., Engrs., U. S. A., Aug. 31, 1918. Overseas service Nov. 12, 1917–July 5, 1919. Discharged July 28, 1919. With 308th and 20th Engrs.; Gen. Staff, G. H. Q., A. E. F. Citation from Gen. Pershing

PILLSBURY, GEORGE BIGELOW
Entered service June 15, 1896; Maj., C. of E., U. S. A., at declaration of war; Col., Engrs., N. A., Aug. 15, 1917. Overseas service Aug. 7, 1918–May 30, 1919. C. O., 115th Engrs., C. O., 102d Engrs.; Corps Engr., 2d Corps.; Office, Chf. Engr., A. E. F.

PIODA, ALBERT WOODBRIDGE
Entered service Oct., 1918; Capt., Engrs., U. S. A., Oct. 23, 1918. Trans. Officer, and Camp Adj., Camp Humphreys.

PIRNIE, HERBERT MALCOLM
Entered service March 7, 1918, as 1st Lt., T. C., N. A.; Capt., T. C., U. S. A., Oct. 27, 1918. Overseas service Mar. 28, 1918–Mar. 21, 1919. Discharged Mar. 26, 1919. Asst. Engr. of Water Supply, H. Q.; Director Gen. of Transportation, A. E. F.

PLANK, DAVID HARLAN
Entered service Sept. 25, 1917; 1st Lt., Engrs., N. A., Apr. 3, 1918. Overseas service July 6, 1918–Aug. 16, 1919. Discharged Aug. 25, 1919. Adj., 516th Engrs. and Engr. Sub-Post, Gen. Intermediate Storage Depot, Gievres, France; Officer in chg. rock supply, Paris and Intermediate Dist. Road Sec.

PLUMER, HAROLD EDWARD
Entered service Oct. 3, 1918, as Maj., Q. M. C., Constr. Div., U. S. A. Discharged Dec. 31, 1918.

POND, FREDERICK HENRY
Entered service Apr. 27, 1918; 1st Lt., E. O. R. C., Feb. 1, 1918; Capt., Engrs., U. S. A., Apr. 7, 1919. Overseas service Aug. 16, 1918–Aug. 6, 1919. Discharged Aug. 28, 1919. Co. Comdr., 34th Engrs., engr. depot work at St. Nazaire, Nantes, and elsewhere in France.

POOLE, CHARLES ARTHUR
Entered service Aug. 31, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 23, 1918. E. O. T. S., Camp Humphreys; with 150th Engrs., Camp Shelby.

POOLE, JOHN HUDSON
Entered service May 6, 1917; Maj., E. O. R. C., Nov. 15, 1916; Lt. Col., Engr. R. C., July 9, 1918; Col., Engrs., U. S. A., Sept. 19, 1918. Overseas service July 31, 1917–Jan. 21, 1919. Discharged Jan. 24, 1919. Bn. Comdr., 16th Engrs.; Gen. Staff, G. H. Q., A. E. F. Officier, Legion d'Honneur; Commandeur, Ordre de l'Etoile Noire.

POORMAN, ALFRED PETER
Entered service May 13, 1917; Capt., Engrs., N. A., July 11, 1917. Overseas service Oct. 31, 1917–June 28, 1919. Discharged July 17, 1919. Co. Comdr., 25th and 29th Engrs.; Supply Officer, G-2-C, G. H. Q., A. E. F.

PORTER, ELMER ALFRED
Entered service Oct. 16, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 18, 1918. With 403d Engrs.; E. O. T. S., Camp Humphreys.

PORTER, HENRY GEORGE
Capt., San. C., U. S. A.*

PORTER, RALPH WALDO
1st Lt., Engrs., U. S. A.*

POST, ADOLPH JOSEPH
Entered service Oct. 5, 1917, as Pvt., Inf., N. A.; Sgt., 1st Class, Q. M. C., N. A., Dec. 7, 1917; 2d Lt., Q. M. C., Constr. Div., N. A., May 16, 1918. Discharged Sept. 19, 1919. Officer in chg. of roads, eng. and boundary survey, Utilities Div., Camp Devens.

POST, WILLIAM SCHUYLER
Entered service May 24, 1917, as Capt., E. O. R. C.; Maj., Eng. R. C., Aug. 15, 1917. Overseas service July, 1918–June, 1919. Discharged July 15, 1919. Bn. Comdr., 316th Engrs.; on staff Chf. Engr. 1st Army, also 5th Army Corps; with Sec. Engr. Intermediate Sec. West, A. E. F. One star.

POTTER, CHARLES LEWIS
Col., Engrs., U. S. A.*

POWELL, ARCHIBALD OLIN
Entered service Aug. 2, 1917; Maj., E. O. R. C., Apr. 16, 1917; Lt. Col., Engrs., N. A., July 23, 1918. Military Asst. to Dist. Engr., 2d Dist.; Asst. Dept. Engr., Eastern Dept.; Enlisted Personnel Officer, Office, Chf. of Engrs., Washington, D. C.; in chg. Eng. Data Branch, Military Div., Office, Chf. of Engrs.

POWELL, HERBERT JAMES BINGHAM
Entered service Oct., 1915, as Staff Officer, British War Mission in U. S. In America from Nov., 1915, to Jan., 1919. Discharged Feb., 1919. Staff Officer, also executive of the British Govt. and American War Dept. in technical matters for the joint Dept. of Gauges and Standards of the Mission and U. S. Bureau of Aircraft Production. Officer of the Order of the British Empire.

POWELL, THOMAS JETT
Entered service May 12, 1917; Capt., E. O. R. C., Apr. 23, 1917; Maj., Engr. R. C., June 30, 1918. Overseas service May 9, 1918–Aug. 21, 1918. Topographic Officer and Engr. Supply Officer, 305th Engrs.

POWELL, WILLIAM JENNER
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 11, 1917; Maj., T. C., U. S. A., May 2, 1919. Overseas service June 30, 1918–June 30, 1919. Discharged July 16, 1919. Co. Comdr. and Bn. Adj., 66th Engrs.; Personnel Adj., 15th Grand Div., T. C.; Highway Div., Peace Commission; Transportation Guard Service, T. C.

POWELSON, WILFRID VAN NEST
Lt. Comdr., U. S. N.*

PRAEGER, EMIL
Entered service Dec. 27, 1917, as Lt., Jr. Grade, C. E. C., U. S. N.; Lt., C. E. C., U. S. N., July 1, 1918. Discharged June 11, 1920. Public Works Dept., 12th and 3d Naval Dists.

PRATT, ARTHUR HENRY
Entered service May 8, 1917; Capt., E. O. R. C., Jan. 22, 1917; Maj., Engrs., U. S. A., Oct. 31, 1918. Overseas service Oct. 30, 1917–Mar. 23, 1919. Discharged July 25, 1919. Student and Asst. Instr., Ft. Oglethorpe and American Univ.; Co. Comdr., 26th Engrs., front line water supply, and C. O., 26th Engrs.. C. O., 1st Bn., 26th Engrs.; Officer in Chg. Water Supply Sec., Office, Chf. Engr., 2d Army and Chf. Engr., Paris Group; Office, Chf. of Engrs., Troop Div. Training Sec. Diploma from Lt. Gen. Bullard for meritorious service. Two stars.

PRESTON, HARRY LONGYEAR
Entered service Aug. 10, 1918; Capt., Engrs., N. A., July 27, 1918. Overseas service Sept. 25, 1918–June 23, 1919. Discharged July 11, 1919. Co. Comdr. and Supply Officer, 22d Engrs.; Office Engr., Div. of Light Rys., Toul Sector; Meuse-Argonne offensive. One star.

PRICE, PHILIP WALLIS
Entered service May 31, 1917; 2d Lt., F. A., N. A., Aug. 15, 1917. Overseas service July 14, 1918–May 22, 1919. Discharged May 24, 1919. With 3d, 310th and 107th F. A.

PRICE, WILLIAM EDMUND
Entered service July 27, 1918, as Capt., Q. M. C., Const. Div., N. A. Discharged Dec. 23, 1918. Asst. to Const. Q. M., Baltimore Depot Warehouses and Camp Hancock.

PRICHETT, FREDERICK BORRADAILE
Entered service July 25, 1917, as Pvt., Engrs., N. A.; Col., Jan. 1, 1918; 2d Lt., F. A., N. A., Apr. 20, 1918. Overseas service, 1918. With 103d Engrs.; with 103d Trench Mortar Battery; O. T. C., Camp Hancock; C. O., H. Q. Co. and Battery Officer, 109th F. A., 28th Div. Died Sept. 6, 1918, from wounds received in action crossing Vesle River, France.

PRINDLE, FRANKLIN COGSWELL
Rear-Admiral, C. E. C., U. S. N. (*Retired.*)*

PRITCHARD, JOHN CHARLES
Entered service Aug. 15, 1917, as 1st Lt., Engrs., N. A.; Capt., Engrs., N. A., Feb. 14, 1918. Overseas service June 29, 1918–Mar. 12, 1919. Discharged Mar. 18, 1919. With 26th Engrs. serving with 3d Army Corps and 1st Army Corps as Dist. Water Supply Officer; installed water points for constr. Verdun-Conflans Ry.; with 3d Army Corps in front of Fismes; Toul Sector; St. Mihiel and Meuse-Argonne offensives. Two stars.

PROCTOR, RALPH FENNO
Entered service May 28, 1917; Maj., Q. M. C., Cantonment Div., June 19, 1917; Lt. Col., Q. M. C., Constr. Div., U. S. A., Aug. 28, 1918. Discharged July 15, 1919. Const. Q. M., Camp Meade and at Curtis Bay Ord. Depot.

PRUETT, GROVER CLEVELAND
Entered service Nov. 8, 1918, as Capt., Engrs., U. S. A. Discharged Mar. 25, 1919. Constr. work Ft. Keogh, Mont.

PUCKETT, HOMER
Entered service May 17, 1918; Capt., Engrs., N. A., May 7, 1918. Discharged Oct. 30, 1919. Member, Bd. of Officers on Heavy Ponton Equipage; head of bridge development unit, Office, Chf. of Engrs., Washington, D. C.

PUGH, MARSHALL ROGERS
Entered service June 13, 1917; Maj., E. O. R. C., Sept. 9, 1917. Overseas service Dec. 26, 1917–Dec. 29, 1918. Discharged Jan. 9, 1919. Bn. Comdr., 21st Engrs. in U. S. and at Nevers, France, in Toul Sector and St. Mihiel offensive; Post Engrs., Bordeaux Embarkation Camp. One star.

PULLIGNY, JEAN LECLERC de
Entered service Nov. 15, 1914, as Commandant, 1st Pioneers, French Army. Discharged June, 1917. In command of the 1st Group of roadbuilders, French 8th Army. Croix de Guerre.

PUNG, WILLIAM SING-CHONG
Entered service Oct. 3, 1917, as Pvt., Inf., N. A.; Cpl., Inf., N. A., Nov. 9, 1917; Sgt., Inf., N. A., Apr. 30, 1918; Sgt., Engrs., N. A., May 10, 1918; 2d Lt., Engrs., N. A., June 6, 1918; 1st Lt., Engrs., U. S. A., Nov. 4, 1918. Discharged Jan. 15, 1919. With 357th Inf.; Instr. in reconnaissance E. O. T. S., Camp Humphreys.

PURCELL, STEUART
Entered service May 12, 1917; Capt., E. O. R. C., June 23, 1917. Discharged Feb. 18, 1919. Engr. Depot, Hoboken; Member, Traveling Examining Bd. for Engr. Officers; Office, Chf. of Engrs.

QUICK, RAY STEVENS
Entered service Aug. 17, 1918; 1st Lt., Engrs., N. A., Aug. 6, 1918. Discharged Jan. 7, 1919. E. O. T. S., and 3d Engr. Training Regt.

QUILTY, T. FRANK
Entered service Feb. 28, 1918, as Maj., Engr. R. C. Discharged May 29, 1919. Constr. Div.; Asst. to Superv. Q. M., Washington, D. C.; in chg. and as Const. Q. M., completed Army Reserve Depot, Columbus, Ohio.

QUINBY, EDWIN RUFUS
Capt., Engrs., U. S. A., A. E. F.*

QUINLAN, GEORGE AUSTIN
Maj., Engrs., U. S. A., A. E. F.*

QUINN, JOHN IGNATIUS
Entered service May 8, 1917; 1st Lt., E. O. R. C., Apr. 5, 1917. Overseas service Nov. 26, 1917–Mar. 6, 1919. Discharged Mar. 26, 1919.

QUINN, MATTHEW FRANCIS
Maj., Engrs., U. S. A., A. E. F.*

QUINN, THOMAS FRANCIS
Entered service Sept. 25, 1917; 1st Lt., E. O. R. C., Aug. 8, 1917. Overseas service Jan. 22, 1918–May 31, 1919. Discharged July 2, 1919. In training Ft. Leavenworth; with 35th, 15th and 501st Engrs.

RADER, FRANKLIN KEARNS
Entered service Aug. 19, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 29, 1918. With 5th Engr. Training Regt., Camp Humphreys.

RADER, JAMES WILSON
Entered service Jan., 1918, as Pvt., 1st Class, A. S., N. A.; 2d Lt., A. S., Mar. 25, 1918. Discharged Jan., 1919. Instr., attached to Cadet Wing, H. Q. Staff, Kelly Flying Field.

RAKESTRAW, CHARLES LYSANDER
Entered service Sept. 2, 1917; 1st Lt., Engrs., N. A., Nov., 1917. Overseas service Apr. 29, 1918–Sept., 1918, and Jan. 15, 1919–July 7, 1919. Discharged Oct. 24, 1919. Bn. Gas Officer, 4th Engrs.; employed on destruction German ammunition dumps. Three stars.

RANDLE, GEORGE NELSON
Entered service Aug. 31, 1918, as Maj., Q. M. C., Constr. Div., U. S. A. Discharged Dec. 2, 1918. Utilities Officer, Camp Fremont and Camp Cody.

RANDOLPH, JOHN HAMPDEN, JR.
Capt., Engrs., U. S. A.*

RANDOLPH, ROBERT ISHAM
Entered service June 5, 1917, as Capt., E. O. R. C.; Maj., Engr. R. C., Dec. 17, 1917. Overseas service Aug. 5, 1918–Apr. 25, 1919. Discharged Apr. 29, 1919. With 23d Engrs.; organized, and C. O., 535th Engrs., attached to 2d Army; standard gauge ry. constr., St. Mihiel Sector. One star.

RANNEY, ALFRED GARDNER
Entered service July 27, 1917; Capt., C. A. R. C., Nov. 27, 1917. Overseas service Mar. 25, 1918–Jan. 30, 1919. Discharged Feb. 20, 1919. Bn. Comdr., 65th Arty.; St. Mihiel and Meuse-Argonne offensives. Two stars.

RATHJENS, GEORGE WILLIAM
Entered service Apr. 27, 1917; Capt., E. O. R. C., Feb. 23, 1917; Maj., E. O. R. C., Aug. 15, 1917; Lt. Col., Engr. R. C., Jan. 4, 1918. Overseas service Aug. 4, 1917–May 30, 1919. Discharged June 17, 1919. With 313th Engrs.; with 2d Army; staff, Chf. of Engrs. Three stars.

RAYMOND, HERBERT DWIGHT
Entered service Nov. 6, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 12, 1918. E. O. T. S., Camp Humphreys.

REAM, WARD HALL
Entered service 1917, as Capt., E. O. R. C.; on account of youth was rerated as 1st Lt., E. O. R. C. Recommendation for promotion to Capt. forwarded to Headquarters Oct. 2, 1918. Overseas service, 1918. O. T. S., Camp Myers; with 305th Engrs.; Co. Comdr., 305th Engrs. Killed Oct. 4, 1918, in action in France.

REDING, JAMES HANDYSIDE
Entered service July 14, 1917, as Capt., Engrs., N. G. (previous federal service from June, 1916, as Pvt., 2d Lt., and 1st Lt., Engrs., with Ohio N. G.). Overseas service June 23, 1918–Mar. 31, 1919. Discharged May 3, 1919. Student, 2d Corps School and Army School, France; Co. Comdr., 112th Engrs. in Baccarat Sector, Avacourt Sector, Meuse-Argonne offensive, Pannes Sector and Ypres-Lys offensive; Supply Officer, Serbian Mission, American Peace Commission, War Damages Bd. Three stars.

REED, CARL SWEETLAND
Entered service Oct. 14, 1918, as Maj., Ord. Dept., U. S. A.; Lt. Col., Ord. Dept., U. S. A., May 21, 1919; Col., Ord. Dept., U. S. A., Aug. 21, 1919. Discharged Oct. 31, 1919. Ord. Dist. Chf., New York Dist.

REED, HOWARD SAWYER
Entered service Dec. 28, 1917; Capt., Engrs., U. S. A., July 16, 1917. Discharged Oct. 30, 1919. Office Chf. of Engrs., Washington, D. C.; Gen. Staff, Portland, Ore.; Regtl. Adj., 216th Engrs.; Representative Adj. Gen. Dept., Camp Devens.

REED, PAUL LYON
Entered service Nov. 6, 1902, as Lt., Jr. Grade, C. E. C., U. S. N.; Comdr., C. E. C., U. S. N., July 1, 1917. Represented Emergency Fleet Corp., Hog Island constr.; Member Compensation Bd., Navy Dept.

REED, PERCY LAWRENCE
Entered service July 25, 1918, as 1st Lt., Ord. Dept., N. A.; Capt., Ord. Dept., U. S. A., May 29, 1919. Discharged Oct. 31, 1919. Duty at Nitrate Plant No. 2, Muscle Shoals, Ala.; Property Officer and C. O., Nitrate Plant No. 3, Toledo, Ohio.

REEDER, HARRY CALVIN
Entered service Jan. 3, 1918; 1st Lt., Engrs., N. A., Dec. 18, 1917. Discharged Apr. 3, 1918. E. O. T. C., Camp Lee.

REES, BIRD LeGRAND
Entered service Apr. 15, 1918; Capt., Engrs., N. A., Apr. 5, 1918. Discharged May 31, 1919. Constr. Div., Washington, D. C.; cantonment constr., Camp Humphreys; with 5th Engr., Training Regt., Camp Humphreys; Office, Chf. of Engrs., Washington, D. C.

REEVES, CARL HOWELL
Entered service July 19, 1917; Capt., E. O. R. C., Apr. 23, 1917; Maj., Q. M. C., Constr. Div., N. A., Mar. 18, 1918. Discharged June 4, 1919. Engr. Officer in chg. constr. Camp Fremont, Camp Doniphan, Ft. Sill and Camp Humphreys; Supervising Const. Q. M., Coast Defenses, Puget Sound; Const. Q. M., Camp Cody.

REEVES, GLENN STANTON
Entered service Dec. 15, 1917, as Pvt., Engrs., N. A.; Pvt., 1st Class, Engrs., N. A., May, 1918; Cpl., Engrs., U. S. A., Aug. 15, 1918. Overseas service Mar. 30 1918–July 21, 1919. Discharged July 26, 1919. With 23d Engrs. Two stars.

REICHARDT, WALTER FREDERICK
Entered service Aug. 9, 1917; Maj., Q. M. C., Constr. Div., May 12, 1917. Discharged Oct. 11, 1917. Asst. to Const. Q. M., Camp Beauregard; transferred to staff of 39th Inf.

REILLY, CHARLES GILBERT
Entered service May, 1917; 1st Lt., Inf., Aug. 1917; Capt., Inf., U. S. A., Aug., 1918. Overseas service, 1918. O. T. C., Fort Niagara; with 313th Inf., at Camp Meade and in France; Co. Comdr., 313th Inf. Killed Sept. 29, 1918, in action in Argonne Forest, France.

REIMER, ARTHUR ADAMS
Entered service May 8, 1917; Capt., E. O. R. C., Jan. 18, 1917; Maj., E. O. R. C., Aug. 15, 1917. Discharged May 6, 1918. Instr., Officers' Training Camp; with 108th and 305th Engrs.

REIMER, FREDERIC ADAMS
Entered service Apr. 24, 1917, as Capt., E. O. R. C.; Maj., E. O. R. C., May 5, 1917. Discharged Feb. 26, 1918. Organized 1st Bn., 104th Engrs.; topographical survey, Camp Dix.; gen. engr. work, Camp Lee; survey work and gen. camp eng. duty, Camp McClellan.

RENSHAW, ALFRED
Entered service May 11, 1917; Capt., E. O. R. C., Aug. 15, 1917. Overseas service, Mar. 29, 1918–Apr. 5, 1919. Discharged Apr. 12, 1919. With 302d Engrs. Three stars.

RENSHAW, FRANCIS OREA
1st Lt., Engrs., U. S. A.*

RESWICK, SOLOMON
1st Lt., Engrs., U. S. A.*

REYNOLDS, LEO FRANCIS
Entered service Dec. 8, 1917, as Pvt., 1st Class, Aviation Sec., Sig. R. C.; 2d Lt., A. S. A., N. A., June 5, 1918. Discharged Jan. 6, 1920. Ground school, Univ. of Texas; concentration camp, Camp Dick; Instr., Love Field; bombing plane training, Ellington Field.

REYNOLDS, RALPH WHITNEY
Entered service Oct. 1, 1917, as 2d Lt., Engr. R. C.; 1st Lt., Engr. R. C., Jan. 3, 1918. Overseas service, June 28, 1918–Mar. 20, 1919. Discharged Apr. 18, 1919. With 316th, 318th, and 37th Engrs. Aisne-Marne, St. Mihiel, and Meuse-Argonne offensives; Defense Sector. Four stars.

RHOADES, THEODORE ECKFORD
Entered service May 1, 1917; Capt., E. O. R. C., Feb. 22, 1917; Capt., G. S., A. E. F., July, 1918; Maj., G. S., U. S. A., Aug. 22, 1918. Overseas service Jan. 15, 1918–Feb. 1, 1919. Discharged Mar. 2, 1919. Const. Q. M. and Officer in chg. constr., Camp Sherman; Asst. Chf. of Staff, 83d Div.; student officer with 11th British Div.; attached to G–4, G. H. Q., and Paris Group, A. E. F.; Asst. Chf. of Staff, G–4, Advanced G. H. Q., Argonne and Germany. Four stars.

RHODES, ERIC HOUGHTON
Lt., Australian Field Engrs., B. E. F.*

RICE, JOHN TURNER
Entered service Dec. 28, 1917; Capt., E. O. R. C., Apr. 18, 1917; Maj., Engr. U. S. A., Feb. 13, 1919. Overseas service Sept. 20, 1918–July 23, 1919. Discharged July 30, 1919. With 2d Engr. Training Regt. and with 541st Engrs.

RICH, EDWARD DUNBAR
Entered service Aug. 12, 1918; Maj., San. C., U. S. A., Aug. 1, 1918. Discharged Dec. 27, 1918. Director, School of San. Eng., Camp Greenleaf.

RICH, MELVIN S.
Entered service May 12, 1917; 1st Lt., E. O. R. C., June 19, 1917. Overseas service Feb. 4, 1918–Feb. 2, 1919. Discharged Feb. 4, 1919. Cantonment constr.; Office, P. B. and G., Washington, D. C.; Labor Bureau, A. E. F.

RICHARDS, WALTER ALLAN
Entered service May 11, 1917; 2d Lt., Inf., N. A., Aug. 29, 1917; 1st Lt., Inf., N. A., Jan. 1, 1918. Overseas service Apr. 29, 1918–Mar. 16, 1919. Discharged Sept. 10, 1919. With 326th Inf. Distinguished Service Cross. Brigade citation for patrol work. Two stars. Two wounds.

RICHARDSON, FREDERICK HOSEA
Entered service Dec. 27, 1917; Capt., Engr. R. C., Sept. 19, 1917. Overseas service July 10, 1918–July 7, 1919. Discharged Aug. 1, 1919. Co. Comdr. and C. O., 526th Engrs.; in chg. constr., Div. Area No. 18 and No. 3, Advance Sec., A. E. F.; C. O., troops at Longuyon, France; in chg. bldg. constr., Univ. of Beaune, France.

RICHARDSON, JAMES HERBERT
Entered service Sept. 25, 1917; Capt., E. O. R. C., Aug. 15, 1917. Overseas service Jan. 20, 1918–June 2, 1918. Discharged Nov. 29, 1918. Co. Comdr., 306th Engrs., Camp Jackson; student, 2d Corps School, France; Co. Comdr., 5th Engr. Training Regt., Camp Humphreys; on duty, Office, Chf. of Engrs., Washington, D. C., revising engr. manual.

RICHARDSON, REX DENSMORE
Capt., Engrs., U. S. A.*

RICHE, CHARLES SWIFT
Entered service July 1, 1882; Col., C. of E., U. S. A., July 1, 1916. Dept. Engr., Central Dept. and Panama Canal Dept.; river and harbor and fortification work.

RICHMOND, ALLEN PIERCE, JR.
Entered service Aug. 21, 1917; 1st Lt., F. A., N. A., Nov. 27, 1917. Overseas service July 15, 1918–Jan. 22, 1919. Discharged Feb. 8, 1919. With 301st Trench Mortar Battery as Gas Officer (in U. S.), Personnel Adj., and Adj.; special instr., F. A., and routine company duties.

RICHMOND, JULIAN
Entered service Aug. 8, 1918, as Lt., C. E. C., U. S. N. R. F. Overseas service Aug. 27, 1918–Dec. 16, 1918. Released from active service Aug. 8, 1919. Asst. Public Works Officer, Naval Aviation Repair Base, Eastleigh, England; Asst. to Public Works Officer, 3d Naval Dist.

RIDDLE, WILLIAM CATHCART
Entered service Oct. 17, 1917, as Capt., San. C., N. A. Discharged June 20, 1919. Div. San. Engr., 81st Div., Camp Jackson; San. Insp. and Camp San. Engr., Hazlehurst Field.

RIDGEWAY, GEORGE ALLEN
Entered service May 14, 1917; 1st Lt., Engrs., N. A., Sept. 6, 1917. Overseas service, Apr. 14, 1918–June 8, 1919. Discharged July 6, 1919. With 23d Engrs. One star.

RINEY, ARTHUR HERBERT
Entered service May 31, 1917; 1st Lt., E. O. R. C., July 2, 1917; Capt., Engr. R. C., Jan. 21, 1918. Overseas service Feb. 10, 1918–July 6, 1919. Discharged July 25, 1919. With 23d Engrs.; Co. Comdr., 28th Engrs., Toul Sector; St. Mihiel offensive. One star.

RIPLEY, BLAIR
Entered service Apr. 12, 1916, as Lt. Col., 1st Canadian Overseas Ry. Constr. Bn. Overseas service Sept. 15, 1916–Apr. 10, 1919. Discharged June 15, 1919. Mentioned in despatches three times; Distinguished Service Order, Great Britain; Commander of the British Empire.

RIPLEY, THERON MONROE
Entered service Dec. 28, 1917; Maj., E. O. R. C., June 19, 1917. Discharged June 24, 1919. Asst. Const. Q. M., Camp Abraham Eustice; Const. Q. M., Ft. Oglethorpe and adjacent camps.

RISLEY, WARNER IRELAND
Entered service May 8, 1917; Capt., E. O. R. C., Feb. 23, 1917. Overseas service Nov. 26, 1917–June 9, 1919. Discharged July 3, 1919. Co. Comdr., 25th Engrs.; Co. Comdr. and C. O., 504th Engrs., in constr. mechanical bakery at Is-sur-Tille, France, and misc. constr.

RITCHIE, JOHN MILTON
Entered service July 26, 1917; Capt., Q. M. R. C., June 7, 1917; Maj., M. T. C., U. S. A., Aug. 19, 1918; Lt. Col., M. T. C., U. S. A., June 18, 1919. In chg. expediting branch, Constr. Div.; Deputy Chf., Operations Div., M. T. C.; Chf., Operations Div., M. T. C.; represented War Dept. on Council of Natl. Defense, Highways Transport Comm.; Member Bd. of Officers to study the military highway system of the U. S.

RITTER, ROLLIN
Entered service Apr. 13, 1917; Capt., F. A., N. A., Aug. 5, 1917. Overseas service May 17, 1917–May 3, 1919. Discharged May 3, 1919. Co. Comdr., 130th F. A., in Gerardmer Defensive Sector, St. Mihiel and Meuse-Argonne offensives and Verdun Sector.

ROBERTS, HARRY ASHTON
Entered service May 8, 1917; Capt., E. O. R. C., June 22, 1917. Overseas service Sept. 25, 1918–Feb. 15, 1919. Discharged Feb. 19, 1919. Shipping Officer, Engr. Depot, New York; Examining Bd., Office, Chf. of Engrs., Washington, D. C.; Deputy Depot Engr. Officer, Marseilles, France.

ROBERTS, LEO BOND
Entered service June 25, 1917; 2d Lt., E. O. R. C., Apr. 2, 1917; 1st Lt. Engr. R. C., Dec. 31, 1917; Capt., Engrs., U. S. A., Feb. 22, 1919; Maj., Engrs., U. S. A., May 16, 1919. Overseas service Jan. 28, 1918–July 3, 1919. Discharged Jan. 17, 1920. Executive Officer G-2-C, G. H. Q., A. E. F.; with 29th Engrs., producing maps for A. E. F. Citation from Gen. Pershing for meritorious service; Officier d'Academie, Silver Palms. One star.

ROBERTSON, ALEXANDER KING
Entered service Apr., 1915; Lt., 72d Regt. of Canada, June 1, 1915; Lt., Royal Engrs., June 17, 1916; Capt., Royal Engrs., Sept. 22, 1918. Overseas service Mar. 18, 1916–Apr. 9, 1919. Discharged Apr. 9, 1919. Mentioned in despatches by Sir Douglas Haig.

ROBERTSON, FONZIE EUGENE
1st Lt., F. A., U. S. A.*

ROBINSON, EDWARD WILLIAM
Maj., Engrs., U. S. A.*

ROBINSON, ERNEST FRANKLIN
Entered service July 15, 1917, as Capt., Engrs., N. A.; Maj., Engrs., N. A., July 30, 1918. Overseas service May 8, 1918–Sept. 7, 1918. Discharged Apr. 15, 1919. With 102d and 74th Engrs.; at Army Engr. School; Ypres-Lys salient. One star.

ROBINSON, JOHN HARVEY
Entered service Apr. 4, 1918; Pvt., Inf., N. A., Apr. 14, 1918; Pvt., 1st Class, Inf., N. A., July 15, 1918; Cpl., Inf., U. S. A., Sept. 20, 1918. Overseas service May 31, 1918–Jan. 15, 1919. Discharged Mar. 15, 1919. With 314th Machine Gun Bn. Two stars.

ROBINSON, JOHN MASON
Capt., Engrs., U. S. A.*

ROBINSON, WILLIAM ERNEST
Entered service May 30, 1917; 2d Lt., E. O. R. C., July 12, 1917; 1st Lt., Engrs., N. A., July, 1918. Overseas service Sept. 13, 1917–Aug. 20, 1918. Discharged Dec. 19, 1918. Student, Ecole du Genie, Versailles; with 1st Engrs.; Intelligence Office, Office, Chf. of Engrs., Washington, D. C. One star.

ROBSON, FREDERICK THURSTON
Entered service Apr. 12, 1917; Capt., E. O. R. C., July 5, 1917; Maj., Engrs., U. S. A., Mar. 1, 1919. Overseas service Apr. 19, 1918–May 21, 1919. Discharged July 3, 1919. Bn. Adj., 307th Engrs.; Asst. G-1 and Asst. Chf. of Staff, G-1, 82d Div. In Lagney-Toul and Marbach Sectors; St. Mihiel and Meuse-Argonne offensives. Div. citation. Three stars.

ROBSON, RALPH EWART
Entered service May 16, 1917, as Capt., E. O. R. C.; Maj., Engrs., U. S. A., Nov. 5, 1918. Overseas service July 5, 1918–Apr. 16, 1919. Discharged May 26, 1919. Co. Comdr. and Bn. Comdr., 316th Engrs.; Defensive Sector; St. Mihiel, Meuse-Argonne and Ypres-Lys offensives. Belgian War Cross. Four stars.

ROBY, HARRISON GEORGE
Entered service Mar. 8, 1918, as Lt., C. E. C., U. S. N.; Lt. Comdr., C. E. C., U. S. N., June 6, 1919. Released from active service June 10, 1919. Asst. Officer in chg. constr., Army and Navy Bldg., Washington, D. C.; Public Works Officer, Quantico Marine Barracks; Asst. Public Works Officer, 4th Naval Dist.

ROCKENBACH, SAMUEL DICKERSON
Entered service Aug. 1, 1891, as 2d Lt., Cavalry, U. S. A.; Brig. Gen. (Temporary), N. A., June 26, 1918. Overseas service May 28, 1917–Aug. 1, 1919. In command Base Sec. No. 1; Chf., Tank Corps. Distinguished Service Medal; Officier, Legion d'Honneur; Croix de Guerre; Companion of the Bath, Great Britain. Three stars.

ROCKHOLD, JOHN ELLSWORTH
Entered service July 19, 1917, as 1st Lt., Engrs., N. A. Overseas service Oct. 18, 1917–July 5, 1919. Discharged July 12, 1919. With 117th Engrs., Baccarat and Champagne Sectors; sapper duties; in chg. road constr., S. O. S., Tours, France. Two stars.

ROCKWELL, REUBEN LYNN
Entered service May 8, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service Jan. 22, 1918–Jan. 26, 1919. Discharged Jan. 29, 1919. With 309th and 35th Engrs.

ROGERS, HERBERT LINCOLN
Comdr., C. E. C., U. S. N. R. F.*

ROGERS, LESTER CUSHING
Entered service Jan. 5, 1918, as Pvt., 1st Class, F. A., N. A.; 2d Lt., F. A., N. A., July 19, 1918. Overseas service May 23, 1918–Jan. 22, 1919. Discharged Jan. 28, 1919. Student, Saumur Arty. School; student and Instr., Anti-Aircraft Arty. School, Arnouville-les-Gonnesse, France.

ROLLINS, ANDREW PEACH
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service Sept. 25, 1918–May 3, 1919. Discharged May 15, 1920. Training engr. replacement troops; with Tank Corps; Bn. Comdr.

ROPES, ELIHU HARRISON
Entered service May 8, 1917, as Capt., E. O. R. C.; Maj., E. O. R. C., Aug. 14, 1917; Lt. Col., Engrs., N. A., Dec. 31, 1917; Col., Engrs., U. S. A., July 31, 1918. Overseas service May 28, 1918–Sept. 7, 1918. Discharged Jan. 3, 1919. With 305th Engrs. as Bn. Comdr. and C. O. in routine sapper duty.

ROSE, WILLIAM HENRY
Entered service Aug. 30, 1899; Lt. Col., C. of E., U. S. A., at declaration of war; Col. and Brig. Gen. Resigned Apr. 30, 1919. Officer in chg. Gen. Engr. Depot in U. S.; Director of Purchase, Purchase, Storage and Traffic Div. Distinguished Service Medal.

ROSEDALE, JOSEPH JACOB
Entered service Sept. 1, 1917, as Capt., Engrs., N. A. Discharged Dec. 17, 1917.

ROSENFELD, JAMES ROY
Entered service July 15, 1917, as Color Sgt., F. A., N. A.; 2d Lt., Engrs., N. A., Aug. 11, 1917. Overseas service June 22, 1918–Mar. 19, 1919. Discharged Mar. 29, 1919. With 107th F. A.; duty with Const. Q. M., Camp Merritt; with 305th and 26th Engrs.

ROSHER, EDWARD MARSHALL
Entered service Apr. 28, 1915, as 2d Lt., Inf., British Army; Capt., Inf., British Army, Aug. 9, 1915; Maj., Rys., Sept. 1, 1917. Overseas service June 14, 1915–Dec. 31, 1919. Discharged Jan. 3, 1920. With 8th Bn., Welsh Regt. Pioneers, 13th Div. at Gallipoli; with rys. force in Mesopotamia and Siberian expedition; Staff Capt., and Ry. Transport Officer; Deputy Asst. Director Rys. Mission to Siberia; Chf. Engr., Bagdad West Div.; British representative for the Interallied Ry. Sub-Comm. Khaborovsk, Siberia; British representative of British Mission, Khaborovsk. Military Cross. Mentioned in despatches, Mesopotamian Expeditionary Forces, and Mediterranean Expeditionary Forces (twice).

ROSS, BLAIR ARTHUR
Entered service May 10, 1917; 2d Lt., E. O. R. C., July, 1917; 1st Lt., E. O. R. C., Aug. 15, 1917; Capt., Engrs., U. S. A., Sept. 1918. Overseas service Sept. 11, 1917–Mar. 11, 1919. Discharged July 8, 1920. C. O., 1st Observation Sec., A. E. F.; Co. Comdr., 29th Engrs. Citation from Gen. Pershing; Officier d'Academie. Three stars.

ROSS, JAMES GEORGE
Entered service May 14, 1917; Capt., Engrs., N. A. Overseas service July 31, 1918–June 15, 1919. Discharged July 8, 1919. Co. Comdr., 306th Engrs.; St. Die and Sommedieu Sectors, and Meuse-Argonne offensive. Two stars.

ROSS, LLOYD McCREIGHT
Entered service July 13, 1918; Pvt., Q. M. C., U. S. A., Sept., 1918; Pvt., Graves Registration Bureau, Nov., 1918. Overseas service Oct. 26, 1918–July 19, 1919. Discharged July 27, 1919. With 336th Supply Co., Q. M. C.; grave location in Meuse-Argonne and St. Mihiel Sectors.

ROSS, THOMAS ALEXANDER
Entered service Mar. 27, 1917, as Capt., Royal Engrs., British Army. Discharged Dec. 12, 1918. In eng. work in War Office, England.

ROSSELL, PAUL FRANCIS
Entered service May 8, 1917; 1st Lt., E. O. R. C., Apr. 2, 1917; Capt., Engr. R. C., Jan. 10, 1918. Overseas service Mar. 30, 1918–May 6, 1919. Discharged May 23, 1919. With 23d Engrs.; Instr. in mining and demolition, Officers Training Camp; investigation of explosives for demolition, Office Chf. of Engrs., Washington, D. C., roads, railroad and bldg. constr., A. E. F.

ROTHROCK, WILLIAM POWELL
Entered service May 11, 1917; Capt., Engrs., N. A., July 24, 1917; Maj., Engrs., N. A., Feb. 15, 1918. Discharged Aug. 1, 1919. Const. Q. M. and Disbursing Officer in chg. constr., Camp Logan;, in chg. constr. two helium plants at Ft. Worth and one at Petrolia, Tex.; picric acid plants Brunswick, Ga., and Grand Rapids, Mich.; Asst. to Chf. of Constr. Div., Washington, D. C.

ROURKE, JOSEPH ALOYSIUS
Entered service Oct. 4, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Jan. 3, 1919. Constr. Camp Devens and South Boston Army Base.

ROUSSEAU, HARRY HARWOOD
Entered service Sept., 1898, as Lt., Jr. Grade, C. E. C., U. S. N.; Rear Admiral, C. E. C., U. S. N., Mar. 4, 1915. With Emergency Fleet Corp. in various capacities, including Asst. Gen. Mgr., Mgr. of Div. of Shipyard Plants, Chairman of Ship Protection Comm.; Project Mgr., Navy Emergency Plant Extension, Bureau of Yards and Docks, Navy Dept.; Member Munitions Bd., Council of National Defense. Navy Cross.

ROWE, DONALD HEFLEY
Entered service Sept. 1, 1917, as 2d Lt., Engrs., N. A.; 1st Lt., Engrs., N. A., Jan. 1, 1918; Capt., Engrs., U. S. A., Oct. 1, 1918. Overseas service Jan. 15, 1918–Apr. 1, 1919. Discharged May 1, 1919. With 314th Engrs., Div. Gas Officer, 81st and 82d Div.; with British Army in Flanders; St. Mihiel and Meuse-Argonne offensives; St. Die and Toul Sectors. Two wounds. Two stars.

ROWLAND, JAMES WALLACE
Entered service May 15, 1918, as Capt., Engrs., N. A. Discharged Feb. 13, 1919. With 4th Engr. Training Regt. as Bn. Adj., Camp Humphreys; Adj., 214th Engrs.

ROYAL, JOSEPH NELSON
2d Lt., Engrs., U. S. A.*

ROYALL, EDWARD MANLY
Entered service Jan. 28, 1918; Lt., Jr. Grade, U. S. N. R. F., Mar. 12, 1917; Lt., Jr. Grade, C. E. C., U. S. N. R. F., Mar. 26, 1918. Released from active service Mar. 12, 1919. Outside Supt., Public Works Dept., Charleston Navy Yard and New Orleans Naval Sta.; gen. duty, 8th Naval Dist.

ROYER, ROBERT STUART
1st Lt., Engrs., U. S. A.*

RUGGLES, ARTHUR VALENTINE
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service Jan. 20, 1918–Jan. 24, 1919. Discharged Jan. 29, 1919. With 111th Engrs.; student, 2d Corps School, France; with 17th Engrs., water supply work, Base Sec. No. 1, A. E. F.

RUSSELL, CLAUD
Entered service June 17, 1918, as Capt., Engrs., N. A. Discharged Dec. 3, 1918. Officers Training School, Camp Humphreys.

RUSSELL, WILLIAM HEPBOURNE
Entered service Aug. 27, 1917; 1st Lt., F. A., N. A., Nov. 27, 1917; Capt. F. A., U. S. A., May 22, 1919. Overseas service June 3, 1918–June 19, 1919. Discharged July 24, 1919. With 324th Regt., F. A., and 158th F. A. Brig. in Argonne forest; 3d Army; Instr. Army Arty. School.

RUST, HENRY PRESTON
Entered service June, 1918, as Lt., Canadian Engrs. Discharged Jan., 1919.

RUTH, EDGAR KINGSBURY
Entered service May 18, 1918, as Capt., Engr. R. C.; Capt. C. W. S., N. A., July 18, 1918. Discharged Dec. 27, 1918. Chf. Gas Officer, Ft. Sill.

RUTTAN, HENRY NORLANDE
Brig. Gen., Canadian Army.*

RYAN, WALTER JACKMAN
Entered service Oct. 17, 1917, as Capt., Engrs., N. A. Discharged Dec. 24, 1917. Officers Training School, Camp Humphreys.

RYLAND, REIS JOSEPH
Entered service Sept. 10, 1917, as Pvt., Sig. C., N. A.; 2d Lt., Sig. C., N. A., Apr. 20, 1918. Overseas service Aug. 3, 1918–Apr. 7, 1919. Discharged Apr. 14, 1919. With 8th Field Bn., Sig. C.; Asst. Div. Sig. Officer, 7th Div.; 2d Army Defense Sector.

RYON, HENRY
Entered service Oct. 15, 1918, as 1st Lt., Q. M. C., Constr. Div., U. S. A.; Capt., Q. M. C., Constr. Div., U. S. A., May 27, 1919. Discharged Oct. 20, 1919. Officer in chg. of water, sewers and electricity; Purchasing and Contr. Officer, Camp Bowie.

SACKETT, ARTHUR JOHNSON
Entered service Jan. 4, 1918, as Maj., Q. M. C., N. A. Discharged Jan. 10, 1919. Constr. camps and port terminals.

SACKS, SAMUEL ISAIAH
Entered service July 26, 1918, as Apprentice Seaman, U. S. N. R. F.; Seaman, 2d Class, Oct. 9, 1918. Released from active service Dec. 4, 1918. Great Lakes Naval Training Sta.; Philadelphia Navy Yard.

SADLER, CARL LEON
Entered service June 25, 1917, as Capt., E. O. R. C. Discharged Mar. 28, 1919. Military mapping at various forts and army camps.

SADLER, WALTER CLIFFORD
Entered service June, 1917, as 2d Lt., E. O. R. C.; Capt., Engrs., U. S. A., Aug. 31, 1918. Overseas service Aug. 9, 1917–May 1, 1919. Discharged May 21, 1919. With 18th Engrs. on dock and gen. constr. work, Base Sec. No. 2, A. E. F.

ST. JOHN, ROYAL UPSON
Entered service Sept. 1, 1917; 2d Lt., A. S., N. A., Nov. 22, 1917; 1st Lt., A. S., U. S. A., Nov. 6, 1918. Overseas service Jan. 30, 1918–Mar. 17, 1919. Discharged Mar. 19, 1919. Operations Officer, 95th Aero Squadron and 2d Pursuit Group; Flying Instr., 2d Aviation Instruction Center. Citation, St. Mihiel and Meuse-Argonne offensives.

SALISBURY, ALFRED JAMES, JR.
Entered service Feb. 24, 1918, as 1st Lt., A. S. A., N. A. Discharged Dec. 4, 1918.

SAMPLE, WILLIAM DWIGHT
Entered service Nov. 23, 1914, as Pvt., Inf., British Army; Sgt., Inf., Dec., 1917. Overseas service July, 1915–Apr., 1918. Discharged June, 1919. Rifle Brig., British Army. Two wounds.

SANGER, WALTER MAX
Entered service Sept. 26, 1917; Capt., Engrs., N. A., Aug. 31, 1917. Overseas service Jan. 25, 1918–July 17, 1919. Discharged Aug. 7, 1919. Gen. const. at Gievres, France. Intermediate Supply Depot; sewer and water layouts for hosp. constr., England; with U. S. Peace Comm. on war damages to Allies; Instr., reinforced concrete design, Beaune Univ.

SARGENT, EDWARD HAYNES
Entered service May 15, 1917, as Capt., E. O. R. C. Overseas service Jan. 24, 1918–Jan. 9, 1919. Discharged Jan. 11, 1919. Co. Comdr. and Regtl. Adj., 20th Engrs.; Co. Comdr. and Bn. Adj., 116th Engrs.; staff duty with Technical Bd., Gen. Purchasing Agent, A. E. F.

SARGENT, JOSEPH ANDREWS
Entered service Sept. 11, 1917, as Capt. Engrs., N. A. Overseas service, Sept. 11, 1917–Mar., 1919. Discharged Feb. 27, 1919. Depot Engr., Gievres, France; with 2d Engrs. as student officer with French, in Bois Belleau, St. Mihiel and Meuse-Argonne. Croix de Guerre; Division Citation. Wounded at Lucy le Bocage, June 2, 1918.

SAUNDERS, WALTER BOWEN
Entered service Nov. 3, 1918, as Capt., Engrs., U. S. A. Discharged Feb. 15, 1919. With 417th Engrs., Camp Dodge; Training School, Camp Humphreys.

SAURBREY, HENRY ALEXIS d'ORIGNY
Entered service Aug. 21, 1918, as Capt., Engrs., U. S. A. Discharged Jan. 17, 1919. With 140th Engrs.; 2d Engr. Training Regt., Camp Humphreys.

SAWYER, CHARLES LEGLER
1st Lt., Engrs., U. S. A.*

SAWYER, DONALD H.
Entered service June 6, 1917, as Maj., Q. M. C.; Lt. Col., Q. M. C., Constr. Div., N. A., Mar. 20, 1918. Discharged Oct. 30, 1919. In chg. constr., Camp Grant, U. S. Nitrate Plant No. 4, and Camp Bragg; in chg. as Superv., Q. M. Warehouses.

SCAMMELL, JOHN KIMBALL
Entered service Nov. 4, 1915, as Lt., Inf., Canadian Forces. Overseas service June 20, 1917–Feb. 14, 1918. Discharged Dec. 12, 1918. Machine Gun Officer, 140th Bn., C. E. F.; C. O., Machine Gun Draft, St. Johns, New Brunswick; Canadian Machine Gun Corps, Seaford, England; C. O., Absentee Depot, New Brunswick.

SCHANCK, FRANCIS RABER
Entered service Nov. 12, 1917; Capt., Ord. Dept., N. A., Nov. 9, 1917; Maj., Ord. Dept., N. A., July 25, 1918. Discharged July 16, 1919. Organized, and Chf., Chicago Dist., Production Div.

SCHARFF, MAURICE ROOS
Entered service Oct. 19, 1917; 1st. Lt., E. O. R. C., Feb. 1, 1917; Capt., Engrs., U. S. A., Nov. 4, 1918. Overseas service Nov. 2, 1917–Mar. 31, 1919. Discharged Apr. 2, 1919. With Water Supply Sec., Div. of Constr. and Forestry, A. E. F.; Water Supply Officer, Office, Sec. Engr., Base Sec. No. 2; Asst. to Deputy Chf. Engr., A. E. F.

SCHEIDENHELM, FREDERICK WILLIAM
Entered service May 14, 1917; Capt., E. O. R. C., Jan. 19, 1917; Maj., Engrs., U. S. A., Sept. 21, 1918; Lt. Col., Engrs., U. S. A., Oct. 21, 1918. Overseas service Oct. 16, 1917–Mar. 12, 1919. Discharged Aug. 19, 1919. With 26th Engrs., front line water supply; Chf., Water Supply Service, 1st Army; C. O., 26th Engrs. Citation from Gen. Pershing for meritorious service.

SCHENCK, ERNEST EUGENE
Entered service Oct. 11, 1918, as 1st Lt., Engrs., U. S. A. Discharged Jan. 12, 1919.

SCHERMERHORN, RICHARD, JR.
Entered service Dec. 4, 1917; Capt., Engr. Sec., San. C., N. A., Nov. 21, 1917. Overseas service Aug. 24, 1918–Apr. 5, 1919. Discharged Apr. 12, 1919. San. Engr. and Asst. to San. Insp., 81st Div., Camp Jackson; C. O., San. Squad No. 1, 87th Div.; Group San. Officer, Justice Hosp. Group, Toul, France; under Gen. McKinstry with Eng. Dept. of Peace Commission, Paris. Two stars.

SCHMUCKER, BEALE MELANCTHON
Entered service Apr. 26, 1917; 1st Lt., N. J. Engrs., Apr. 30, 1917; 1st Lt., Engrs., N. A., Aug. 5, 1917; Capt., Engrs., U. S. A., May 2, 1919. Overseas service June 19, 1918–May 22, 1919. Discharged June 5, 1919. With 104th Engrs. in U. S. and overseas; Meuse-Argonne offensive; Haute-Alsace Sector. One star.

SCHODER, ERNEST WILLIAM
Entered service Sept. 25, 1917; Capt., Engrs., N. A., Aug. 15, 1917. Discharged Apr. 2, 1919. With 305th Engrs. at Camp Lee; Instr., E. O. T. S., Camps Lee and Humphreys; Asst. Dir. Training, Camp Humphreys; Office, Chf. of Engrs.

SCHOLTZ, HERMAN FRED
Entered service June 20, 1918; Capt., Engr. R. C., Jan. 28, 1918. Overseas service Oct. 19, 1918–July 14, 1919. Discharged Aug. 14, 1919. E. O. T. S., Camps Lee and Humphreys; Co. Comdr., 1st Bn., Camp Forrest; Co. Comdr., 128th Engrs.

SCHOONMAKER, LEON MONROE
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 5, 1917. Overseas service Feb. 20, 1918–June 5, 1919. Discharged June 19, 1919. Training Camp, American Univ.; with 105th Engrs., Camp Sevier; Co. Comdr., 511th Engrs., in gen. engr. and constr. work, Montierchaume Storage Depot, France.

SCHROEDER, SEATON, JR.
Entered service Apr. 6, 1917, as Lt., Jr. Grade, U. S. N. R. F.; Lt., C. E. C., U. S. N. R. F., Dec. 10, 1918. Released from active service June 6, 1919. In chg. constr. work in Brooklyn Navy Yard and Navy Fleet Supply Base, Brooklyn, N. Y.

SCHWARTZ, LLOYD
Entered service June 26, 1918, as Pvt., C. A. C., U. S. A.; Master Gunner, C. A. C., U. S. A., Feb. 15, 1919. Overseas service Aug. 7, 1918–May 23, 1919. Discharged June 21, 1919. Portland Coast Defenses; with 72d Regt., C. A. C.

SCHWENDENER, KARL DE WITT
Entered service May 8, 1917; Capt., E. O. R. C., Apr. 16, 1917. Overseas service Aug. 8, 1918–June 28, 1919. Discharged July 24, 1919. Regtl. Supply Officer and Bn. Comdr., 115th Engrs.; Marbache and Toul Sectors; Meuse-Argonne offensive. Two stars.

SCOTT, GUY
Entered service May 15, 1918, as Capt., Engrs., N. A. Overseas service July 30, 1918–July 5, 1919. Discharged July 24, 1919. Personnel and Engr. Officer, and Co. Comdr., 44th Engrs.; Div. Engr. Etat Ry., France; Engr., M. of W., 16th Grand Div., T. C., A. E. F.

SCOTT, LEWIS PELOT
Entered service Aug. 17, 1918; 1st Lt., Engrs., U. S. A., Aug. 7, 1918. Discharged Oct. 10, 1918. E. O. T. S., Camp Humphreys.

SCUDDER, CHARLES MORRISON
Entered service July 15, 1917, as Capt., Engrs., N. A. Overseas service Jan. 31, 1918–May 18, 1919. Discharged May 27, 1919. Co. Comdr. and Topographic Officer, 107th Engrs.

SCUDDER, HENRY DARCY, JR.
Entered service May 24, 1918, as 2d Lt., U. S. Guards. Discharged Jan. 29, 1919. Guard duty in the U. S.

SEABURY, GEORGE TILLEY
Entered service Apr. 6, 1918, as Maj., Q. M. C., N. A. Discharged June 14, 1919. Asst. to Chf. of Constr. Div., as Superv. Const. Q. M. at Camps Devens, Upton, Mills, Merritt, Dix, Meade and Lee.

SEAGE, CLARENCE EDMUND
Entered service June 27, 1917, as Ensign, U. S. N. R. F.; Lt., Jr. Grade, U. S. N. R. F., Jan. 1, 1918; Lt., U. S. N. R. F., July 1, 1918. Released from active service Jan. 4, 1919. Recruiting duty, Staten Island; in charge constr. Naval Base, Tarrytown, N. Y.; on duty with Ord., C. N. A., as Insp., mine development; U. S. S. *North Carolina* on convoy duty in foreign waters; attached Naval Unit, Univ. Penna., as Executive Officer.

SEARIGHT, GEORGE PETER
Entered service Dec. 28, 1917. 1st Lt., Engr. R. C., Sept. 24, 1917; Capt., Engrs., U. S. A., Feb. 26, 1919. Overseas service June 30, 1918–Mar. 20, 1919. Discharged Apr.

18, 1919. With 302d Engrs., Aisne-Marne offensive; with 27th Engrs., Army troops in St. Mihiel and Meuse-Argonne offensives. Four stars.

SEELY, RAY
Entered service June 10, 1918; Capt., Engrs., U. S. A., Sept. 1, 1918. Discharged Dec. 10, 1918. In training, Camp Humphreys; Co. Comdr., 10th Engr. Training Regt.

SEELYE, SETH HENNESS
Pvt., Engrs., U. S. A., A. E. F.*

SELANDER, JOHN EINER
Maj., Royal Engrs., B. E. F.*

SELL, WILLIAM DRUMM
Entered service Mar. 1, 1917, as Capt., E. O. R. C. Discharged Oct. 23, 1917. In training Ft. Benjamin Harrison and Ft. Leavenworth.

SENIOR, FRANK SEARS
Entered service Jan. 26, 1918, as Maj., Engrs., N. A. Overseas service Feb. 15, 1918–May 31, 1919. Discharged June 3, 1919. Engr. of Ports, staff of Director Gen. of Transportation, A. E. F. Officier d'Academie; Citation from Gen. Pershing for meritorious service.

SESSLER, GROVER CLEVELAND
Entered service July 12, 1918, as Ensign, C. E. C., U. S. N. R. F.; Lt., Jr. Grade, C. E. C., U. S. N. R. F., Apr. 1, 1919. Released from active service June, 1919. On constr. Army and Navy Bldgs., Washington, D. C.; in chg., labor, yards, and docks contracts.

SEWARD, OSCAR A., JR.
Entered service May 8, 1917; Capt., Engrs., N. A., Aug. 15, 1917. Overseas service June, 1918–June, 1919. Discharged July 16, 1919. Bn. Adj. and Bn. Comdr., 315th Engrs. Three stars. One wound.

SEWELL, JOHN STEPHEN
Entered service Jan. 23, 1917, as Maj., E. O. R. C.; Col., E. O. R. C., July 6, 1917. Overseas service July 28, 1917–July 26, 1919. Discharged Aug. 14, 1919. C. O., 17th Engrs. and Sec. Engr., Base Sec. No. 1, A. E. F.; C. O., Base Secs. No. 1 and No. 9, A. E. F. Distinguished Service Medal; Officier, Legion d'Honneur; Officer, Order of Leopold, Belgium.

SEXTON, GEORGE FRANCIS
Entered service Nov. 25, 1917; Pvt. to Sgt., 16th Co., 4th Training Bn., Nov. 7, 1917 to Jan. 15, 1918; Pvt. to Sgt. 1st Class, Constr. Div., N. A., Jan. 15 to May 1, 1918; 2d Lt., Q. M. C., Constr. Div., N. A., May 1, 1918. Discharged May 7, 1919. Camp Jackson; constr. work at Columbia, S. C., Baltimore, Md., and Washington, D. C.

SEYMOUR, HORATIO
Entered service Feb. 7, 1918, as 1st Lt., Ord. C., N. A.; Capt., Q. M. C., Constr. Div., U. S. A., Sept. 4, 1918. Discharged July 19, 1920. Constr. Sec., Supply Div., Ord. Dept. on constr. work at Curtis Bay and Raritan Arsenals; Constr. Q. M., Naco, Ariz.

SHAFER, ERNEST ALTON
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., June 19, 1917. Overseas service Mar. 30, 1918–Mar. 23, 1919. Discharged Apr. 30, 1919. Engr. Replacement Troops Depot; with 26th Engrs.

SHAFER, JAMES CHARLES FORSYTHE
Entered service Aug. 27, 1917; Capt., C. A. C., N. A., Nov. 27, 1917; Maj., C. A. C., U. S. A., Nov. 2, 1918. Overseas service Dec. 12, 1917–Nov. 24, 1918. Discharged Dec. 11, 1918. Battery Comdr., 52d Arty., C. A. C., at Thierville, Les Sartelle, St. Mihiel and Meuse-Argonne. Two stars.

SHANER, HARRY LINDEN
Entered service Sept., 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Dec., 1919. Officer in chg. Water and Sewers, Camp Greene; Utilities Officer, Camp Gordon.

SHANKLAND, RALPH GRAHAM
1st Lt., Engrs., U. S. A., A. E. F.*

SHAW, ARTHUR LASSELL
Entered service May 14, 1917; Capt., E. O. R. C., June 11, 1917. Overseas service July 14, 1918–June 13, 1919. Discharged July 8, 1919. Bn. Adj., Co. Comdr., and Regtl. Adj., 301st Engrs. as Div. Engrs. and Corps Engrs., 4th Army Corps, St. Mihiel offensive, Toul Sector and Army of Occupation. Two stars.

SHAW, ARTHUR MONROE
Entered service July 20, 1917; Maj., E. O. R. C., June 19, 1917. Discharged June 3, 1919. Const. Q. M., Camp Beauregard, Camp Jessup, and Camp Normoyle; Asst. to Chf. of Constr. Div., Washington, D. C.

SHAW, FRANKLIN DICKINSON
Entered service Sept. 24, 1917, as Capt., Engrs., N. A.; Maj., Q. M. C., Constr. Div., U. S. A., Apr. 28, 1919. Asst. Supervisor, Supervisor and Sec. Chf. on bldg. projects for Ord. Dept.

SHEA, WILLIAM EDWARD
Entered service Nov. 7, 1918. Discharged Nov. 21, 1918. E. O. T. S. Camp Humphreys.

SHEARER, DAVID McDOUGALD
Entered service May 14, 1917; Capt., E. O. R. C., July 16, 1917. Overseas service, Sept. 1, 1918–Sept. 5, 1919. Instr. 2d and 3d Engr. Officers Training Camps; Regtl. Supply Officer, 604th Engrs.; Asst. G-1, 1st Div., A. E. F.; Meuse-Argonne offensive. One star.

SHEIBLEY, EDWARD GWYN
Entered service May 1, 1917; 1st Lt., E. O. R. C., Feb., 1917. Discharged June 29, 1917.

SHELEY, HORACE WEST
Entered service Dec. 28, 1917; Capt., E. O. R. C., Sept. 27, 1917. Overseas service Oct. 26, 1918–July 16, 1919. Discharged July 29, 1919. Engr. Observer in aerial mapping; training troops, Camp Humphreys; Co. Comdr., 548th Engrs; with 20th Engrs. in Advance Sec., A. E. F.; Dist. Road Engr., S. O. S., A. E. F. One star.

SHEPARD, GEORGE MILSON
Entered service May 9, 1917; Capt., E. O. R. C., May 26, 1917. Service in Hawaiian Territory Feb., 1918–Sept., 1918. Discharged Jan. 10, 1919. Engr. Instr., Inf. Officers Training School, Ft. Snelling; Co. Comdr., 3d Engrs., Hawaii; Bn. Comdr., 606th Engrs., Camp Humphreys.

SHEPPARD, NORMAN KIRKWOOD
Entered service Sept. 4, 1917, as Pvt., Engrs., N. A.; through all grades to Capt., Engrs., U. S. A. Service in Philippine Islands Jan. 6, 1919–Feb. 25, 1920. Discharged Feb. 25, 1920. With 313th Engrs.; Co. duty and in chg. constr. work.

SHERIDAN, LAWRENCE VINNEDGE
Entered service Jan. 5, 1918, as Pvt., 1st Class, F. A., N. A.; Sgt., F. A., N. A., Apr. 22, 1918; 2d Lt., F. A., N. A., July 10, 1918. Overseas service May 23, 1918–Jan. 22, 1919. Discharged Jan. 28, 1919. With 324th Heavy Arty.; Saumur Arty. School and Anti-Aircraft School, Angers, France.

SHERMAN, EDWARD CLAYTON
Entered service Sept. 11, 1918, as Lt. Comdr., C. E. C., U. S. N. R. F.; Comdr., C. E. C., U. S. N. R. F., June 26, 1919. Project Mgr. Bureau of Yards and Docks, Navy Dept., Washington, D. C. Officier de l'Academie. One star.

SHERRON, GEORGE AUSTIN
Entered service Sept. 2, 1917, as Capt., E. O. R. C. Resigned Dec. 16, 1917. With 115th Engrs.

SHIBLEY, KENNETH
Entered service Oct. 22, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 10, 1918. With 403d Engrs.

SIBERT, WILLIAM LUTHER
Entered service 1880; Maj. Gen., U. S. A., June, 1917. Overseas service June, 1917–Jan., 1918. Now on retired list. C. O., 1st Div.; Director, C. W. S. Distinguished Service Medal; Commandeur, Legion d'Honneur. One star.

SILSBEE, JAMES ALFRED
Entered service Jan., 1918, as 1st Lt., Ord. R. C.; 1st Lt., C. W. S., U. S. A., July, 1918. Discharged July 8, 1920. Constr. and operation poison gas plant, Englewood Arsenal; Co. Comdr., 2d Bn.; C. O., Charleston Plant, Edgewood Arsenal.

SILVERNAIL, ALFRED KIMBERLY
Entered service Dec. 6, 1917, as Pvt., Medical C., N. A.; 1st Lt., Q. M. C., Constr. Div., N. A., Mar. 23, 1918; Capt., Q. M. C., Constr. Div., U. S. A., Aug. 24, 1918. Discharged Aug. 20, 1919. In chg. constr. work at Edgewood Arsenal, Springfield, Mass., Armory, Frankfort Arsenal, and Rock Island Arsenal.

SJOVALL, ARVID HENRY
Entered service Feb. 13, 1918, as Capt., Q. M. C., Constr. Div., N. A. Officer in chg. Water Supply and Sewers at Camps Devens, Custer and Grant; Utilities Officer, Camp Devens.

SKELLY, JAMES WILLIAM
Entered service May 8, 1917; Capt., E. O. R. C., Feb. 11, 1917; Maj., Engrs., U. S. A., Mar. 13, 1919. Overseas service July 28, 1917–Apr. 27, 1919. Discharged May 31, 1919. With 12th Engrs. as Regtl. Supply Officer; Office, Chf. Engr., 3d Army. Military Cross, Great Britain, for work in Somme defensive. Five stars.

SKINNER, JOHN FRANKLIN
Entered service Sept. 16, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Apr. 15, 1919. Supervisor of Utilities in army camps and cantonments in dist. north of North Carolina and east of Mississippi River; member, bd. to advise and report on water supply and other utilities in Western Dept. and Hawaii.

SKINNER, THEODORE HOBART
Entered service June 2, 1917; Capt., E. O. R. C., Feb. 7, 1917. Released from active service Mar. 8, 1918. Field Engr. in Camp Dix survey; Const. Q. M., Camp Dix; Asst. to Officer in Chg. Cantonment Div.

SLEIGHT, REUBEN BENJAMIN
Entered service May 8, 1917; 2d Lt., Aviation Sec., Sig. R. C., Jan. 16, 1918; 1st Lt., A. S. A., U. S. A., Sept. 25, 1918. Released from active service Aug. 19, 1919. Production statistics, organization and tonnage estimates for Air Service material; Chf. of Tonnage Sec., Bureau of Aircraft Production.

SLEPPY, KIRBY BALDWIN
Entered service 1917; Capt., E. O. R. C., Aug. 15, 1917. Overseas service 1917–1918. Co. Comdr., 4th Engrs. Killed Aug. 4, 1918, in action at Chery Chartreuve, France.

SLIFER, HIRAM JOSEPH
Entered service Aug. 15, 1917, as Maj., E. O. R. C.; Lt. Col., Engrs., N. A., Sept. 28, 1917. Overseas service 1917–1919. With 311th Engrs.; C. O. 21st Engrs. (Light Ry.). Died Feb. 3, 1919, in active service in France.

SLOAN, WILLIAM GLENN
Entered service Dec. 29, 1917, as 1st Lt., Engrs., N. A. Overseas service, July 8, 1918–Feb. 11, 1919. Discharged Feb. 27, 1919. Topographic mapping, U. S. Geological Survey; Asst. Bn. Supply Officer, 29th Engrs., Langres, France; Executive Asst. to Asst. Chf. of Staff, G–2–C, Chaumont, France.

SLOAN, WILLIAM GRIFFITH
Entered service Feb. 18, 1918, as Maj. Q. M. C., Constr. Div., N. A.; Maj., Engrs., U. S. A., Aug., 1918. Overseas service Aug. 1918–Jan. 1919. Discharged Jan. 10, 1919. Supervising Const. Q. M., N. E. Sec. of U. S.; Bn. Comdr., 22d Engrs., in constr. and maintenance of light rys. on St. Mihiel and Meuse-Argonne fronts.

SLOCUM, CURLYS LYON
Entered service Sept. 23, 1918, as Capt., Ord. Dept., U. S. A. Discharged July 31, 1919. Asst. to C. O., Springfield Armory, Springfield, Mass., Ord. Dept.

SMALL, JAMES HAMPDEN, JR.
Entered service Apr. 11, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged May 6, 1919. In chg. of construction, Richmond Bag Loading Plant, Seven Pines, Va.

SMALLMAN, RALPH ALCORN
Entered service May 14, 1917; 1st Lt., E. O. R. C., July 5, 1917; Capt., Engr. R. C., Dec. 1, 1917. Overseas service Feb. 27, 1918–May 23, 1919. Discharged June 18, 1919. Co. Comdr., 25th Engrs., gen. constr. work in Base Secs. No. 2 and No. 7, A. E. F., and in road work, 1st Army; Meuse-Argonne offensive; constr., Camp Pontenazen. One star.

SMEAD, RAPHAEL CHART †
Maj., Engrs., U. S. A.*

SMILLIE, RALPH
Entered service Aug. 15, 1917, as Ensign, N. N. V.; Lt., Jr. Grade, U. S. N. R. F., Sept. 21, 1918. Released from active service June 21, 1919. Navigating Officer, U. S. S. *Massachusetts* and Navigator, U. S. S. *Bushnell.*

SMITH, ALBERT
Entered service May 14, 1917; Capt., Engr. R. C., July 19, 1917; Maj., Engrs., N. A., Aug. 15, 1917; Lt. Col., Engrs., U. S. A., Aug. 20, 1918. Discharged Apr. 22, 1919. Bn. Comdr., 309th Engrs.; Staff School, War College; 2d in command, 215th Engrs., Camp Logan.

SMITH, ALEXANDER CRAWFORD, JR.
Entered service Sept. 17, 1917; 1st Lt., E. O. R. C., Feb. 12, 1918. Overseas service Mar. 29, 1918–June 6, 1919. Discharged June 11, 1919. With Truck Co. 10, 23d Engrs.; Chateau Thierry, St. Mihiel, and Argonne-Meuse offensive. Three stars.

SMITH, CHARLES EDWARD
Entered service Apr. 24, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged Feb. 13, 1919. Asst. to Executive Officer, Eng. Div., Constr. Div.; visited and reported on proposed cantonment sites; Sec. Engr. and Asst. to Chf. of Sec. A, Bldg. Div., Washington, D. C.; Acting Officer in Chg. Sec. A.

SMITH, CHESTER ALEXANDER
Entered service Apr. 2, 1918, as Capt., San. C., N. A. Discharged Dec. 18, 1918. C. O., San. Detachment, Camp Joseph E. Johnston; Camp San. Engr., Camp McClellan.

SMITH, CHESTER KITCH
Entered service July 7, 1917; 1st Lt., E. O. R. C., June 19, 1917; Capt., Engrs., N. A., May 9, 1918; Maj., Engrs., U. S. A., Oct. 8, 1918. Overseas service Aug. 9, 1917–Feb. 3, 1919. Discharged Feb. 6, 1919. With 18th Engrs. on r. r. and dock constr. in Base Sec. No. 2; Asst. to Chf., R. R. and Dock Sec., Div. of Constr. and Forestry, A. E. F.

SMITH, CLAIRE HOWLAND WALLACE
1st Lt., Engrs., U. S. A., A. E. F.*

SMITH, CLARENCE URLING
Entered service May 11, 1917, as Capt., E. O. R. C. Overseas service Jan. 29, 1918–May 18, 1919. Discharged May 21, 1919. With 107th Engrs.; Engr. Officer in chg. constr. at Langres, France; Supt. of Constr., 64th Brigade Area, 3d Army; Alsace Sector; Aisne-Marne, Oise-Aisne, and Meuse-Argonne offensives. Croix de Guerre. Citation by Gen. Petain. Five stars.

SMITH, CLARKE STULL
Entered service June 20, 1894; Lt. Col., C. of E., U. S. A., May 15, 1917; Col., Engrs., N. A., Aug. 5, 1917. Overseas service Sept. 9, 1918–Apr. 2, 1919. Organized and commanded 311th Engrs.; C. O., 305th Engrs.

SMITH, EARL AUDIE
1st Lt., Engrs., U. S. A.*

SMITH, EDWARD KING
Entered service Aug. 27, 1917; 2d Lt., Sig. C., N. A., Nov. 11, 1917. Overseas service Jan. 13, 1918–Apr. 30, 1919. Discharged May 6, 1919. Sig. C. Photo Laboratory, Paris; Depot Engr. Officer, Sig. C. Replacement Depot, Cour Cheverny, France.

SMITH, EVERETT CLERC, JR.
Entered service May 12, 1918; Capt., Engrs., N. A., June 14, 1918. Overseas service Aug. 16, 1918–Aug. 15, 1919. Discharged Aug. 27, 1919. Operation Gen. Engr. Supply Depot, Gievres, France, with 34th Engrs.

† Died Nov. 28, 1919.

SMITH, FRANCIS MARSHALL
Entered service Aug. 25, 1917; Capt., E. O. R. C., Aug. 23, 1917; Maj., Engrs., U. S. A., Oct. 3, 1918. Overseas service, Jan. 31, 1918–Mar. 19, 1919. Discharged Mar. 25, 1919. Attached to British 36th Div. in Belgium. C. O., Camp de Grasse, Tours, France; C. O., 51st Engrs.; Engr. Officer Inter-Allied Commission, Treves, Germany. Two stars. One wound.

SMITH, GILMAN WALTER
Maj., Q. M. C., U. S. A.*

SMITH, LANDON GARLAND
1st Lt., Engrs., U. S. A., A. E. F.*

SMITH, LAYTON FONTAINE
Entered service Apr. 17, 1917; Lt., Jr. Grade, U. S. N., Jan. 31, 1917; Lt., U. S. N., Dec. 1, 1917. Released from active duty Sept. 23, 1919. Public Works Dept., Charleston Navy Yard; Executive Officer, Receiving Yards, New London, Conn., Naval Base; U. S. S. *Maine*; 1st Watch Officer, U. S. S. *Pretoria*, transport service to Brest.

SMITH, MAXWELL WAIDE
Entered service May 8, 1917, as Capt., E. O. R. C.; Maj., Engrs., N. A., July 30, 1918. Overseas service May 28, 1918–Sept. 12, 1918. Discharged Nov. 28, 1919. Co. and Bn. Comdr., 308th Engrs.; Supt. of Military Ry., Camp Humphreys; Asst. Prof., Military Science and Tactics, Univ. of Illinois. Aisne-Marne offensive. One star.

SMITH, MERRITT HAVILAND
Entered service June 30, 1917; Col., F. A. R. C., Mar. 23, 1917. Overseas service June 30, 1918–Jan. 3, 1919. Discharged Feb. 12, 1919. With 104th F. A. to assist in instruction, Plattsburg; Meuse-Argonne offensive. Two stars.

SMITH, RICHARD BENNETT
Entered service Apr. 25, 1918; Pvt., Sig. C., N. A., May, 1918. Overseas service Sept. 1, 1918–Mar. 7, 1919. Discharged Mar. 29, 1919. Served as observer at adv. meteorological sta. in Meuse-Argonne offensive.

SMITH, ROBERT COLFAX
Entered service Mar. 23, 1918, as Maj., Q. M. C., Constr. Div., N. A. Discharged June 14, 1919. Const. Q. M., Interior Storage Depot, Schenectady, N. Y.; Asst. to Const. Q. M., Chicago Warehouses.

SMITH, SCHUYLER MORTON
Entered service May 20, 1917; 1st Lt., E. O. R. C., May 28, 1917; Capt., Engrs., U. S. A., Nov. 3, 1918. Overseas service July 28, 1917–Feb. 5, 1919. Discharged Feb. 7, 1919. With 12th Engrs.; Office, Chf. Engr., G. H. Q., A. E. F. Three stars.

SMITH, SHALER GORDON
Entered service Aug. 17, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 24, 1918. Asst. Personnel Adj., 8th Engr. Training Regt.; with 5th Engr. Training Regt.

SMITH, WILLIAM ERNEST
Entered service Sept. 21, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 10, 1918. E. O. T. S., Camp Humphreys.

SMITH, WILLIAM WOOD
Capt., Engrs., U. S. A., A. E. F.*

SMOOT, LLOYD DUVALL
Capt., Q. M. C., U. S. A.*

SNODGRASS, WILLIAM TYLER
Entered service Jan. 31, 1918, as Pvt. Engrs., N. A.; through all grades to M. E., Sr. Grade, Engrs., U. S. A., Mar., 1919. Overseas service July 1, 1918–July 5, 1919. Discharged July 18, 1919. With 28th Engrs., quarry and road work, 1st Army. One star.

SNOOK, THOMAS EDWARD, JR.
Entered service May 8, 1917; 2d Lt., E. O. R. C., Mar. 15, 1917; 1st Lt., Engr. R. C., Aug. 15, 1917; Capt., Engrs., U. S. A., May 18, 1918. Overseas service July 30, 1918–June 15, 1919. Discharged July 3, 1919. Co. Comdr. and Bn. Adj., 306th Engrs.; St. Die Sector and Meuse-Argonne offensive. One star.

SNYDER, FREDERIC ANTES
Entered service July 27, 1917; Maj., E. O. R. C., Apr. 18, 1917; Lt. Col., Engrs., N. G., July 25, 1917; Col., Engrs., N. G., Jan. 23, 1918. Overseas service May 18, 1918–May 1, 1919. Discharged June 5, 1919. C. O., 103d Engrs.; Div. Engr., 28th Div.; with 34th Div. (British); with 39th Div. (French) on constr. of fortifications, Chateau Thierry to Conde en Brie. Chateau Thierry defensive; Champagne-Marne defensive; Aisne-Marne offensive; Fismes Sector; Aisne offensive; Meuse-Argonne offensive; Thiaucourt Sector. Distinguished Service Medal. Five stars.

SNYDER, GEORGE DUNCAN
Enterod service July 15, 1917, as Capt., Engrs., N. A.; Maj., Engrs., U. S. A., Feb. 19, 1919. (Capt., 22d N. Y. Engrs. at time of Federalization.) Overseas service May 17, 1918–Mar. 11, 1919. Discharged Apr. 3, 1919. With 102d Engrs.; on War Damages Bd., American Peace Commission, France and Italy.

SNYDER, HUBERT EARL
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., June 19, 1917; Capt., Engr. R. C., Dec. 26, 1917. Discharged Mar. 1, 1919. With 20th Engrs., Camp American Univ.; Purchasing Div., Gen. Engr. Depot, Washington, D. C.

SOEST, HUGO CONRAD
Entered service May 8, 1917; 1st Lt., E. O. R. C., Jan. 26, 1917. Overseas service Oct. 31, 1917–Mar. 17, 1919. Discharged Mar. 20, 1919. With 25th Engrs.; office, Chf. Engr.,

3d Army; Inter-Allied Ry. Comm.; Office, Chf. Engr., G. H. Q., A. E. F.; Meuse-Argonne offensive. One star.

SOLOMON, GABRIEL ROBERTS
Entered service Feb. 20, 1918; Maj., Engrs., N. A., Feb. 16, 1918; Lt. Col., Q. M. C., Constr. Div., U. S. A., Apr. 30, 1919; Col., U. S. A. (unattached), July 3, 1919. Discharged Feb. 4, 1920. Representative, U. S. Smokeless Powder Plant, No. 2, Nitro, W. Va.; Executive Asst., Eng. Div.; Asst. Sec. Chf., Sec. B, Bldg. Div.; Officer in Chg., Eng. Div.; all duties with Constr. Div.

SOMMER, ISADOR MENDELSOHN
Entered service Dec. 15, 1917, as Pvt., Q. M. C., N. A.; Sgt., Q. M. C., N. A., Apr., 1918; 1st Sgt., Q. M. C., N. A., July, 1918. Overseas service Sept. 6, 1918–June 8, 1919. Discharged July 30, 1919. Asst. in charge utilities, Camp Joseph E. Johnston.

SOPER, GEORGE ALBERT
Entered service Jan. 18, 1918, as Maj., San. C., U. S. A. Discharged Sept. 20, 1919. Epidemiologist, Oglethorpe group of camps, Chickamauga Park, Ga., and in Div. of Sanitation, Surgeon General's Office.

SOURWINE, JAMES ARTHUR
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 18, 1917. Overseas service Apr. 29, 1918–Sept. 15, 1919. Discharged Oct. 18, 1919. With 310th Engrs. Camp Custer; Adj. and Supply Officer, Prov. Bn., 16th Engrs.; with 510th Engrs.; Engr. School, Langres, France; with 2d Engrs. in St. Mihiel drive; with Highway Sec., War Damages Bd.; special duty with "Ecole des Ponts et Chaussees", Wounded Sept. 13, 1918, in St. Mihiel drive.

SPEAR, PHILIP HICHBORN
Entered service May 8, 1917, as 1st Lt., E. O. R. C.; Capt., E. O. R. C., Aug. 15, 1917. Overseas service Sept. 24, 1918–Aug. 15, 1919. Discharged Aug. 30, 1919. With 301st Engrs. at Camp Devens; Asst. Depot Engr., New York City; in Personnel Div., Office, Chf. of Engrs.; with various engr. depots in France.

SPEAR, WALTER EVANS
Entered service Oct. 26, 1917, as Maj. Q. M. R. C. Discharged Aug. 5, 1919. Utilities Officer, Camp Upton; Const. Q. M., Camp Upton.

SPENCER, CHARLES BURR
1st. Lt., Ord., U. S. A., A. E. F.*

SPENCER, HERBERT
Entered service Oct. 25, 1917; Capt., Engrs., N. A., Feb. 11, 1918. Overseas service Mar. 30, 1918–June 10, 1919. Discharged June 11, 1919. Co. Comdr., 23d Engrs.; Camp Engr., Camp Coetquidon, A. E. F.; Asst. Corps Roads Officer, 1st Army; in charge road maintenance after armistice in Toul Area. Three stars.

SPERRY, LOUIS NEWTON
Entered service May 8, 1917; 1st Lt., E. O. R. C., Feb. 19, 1917. Overseas service Dec. 11, 1917–Mar. 11, 1919. Discharged Apr. 3, 1919. With 80th Div. and 2d, 6th and 102d Engrs.; Instr., 1st Corps School, A. E. F.

SPRAGUE, HUGH MAX
Entered service Dec. 26, 1917; 1st Lt., Engr. R. C., Oct. 4, 1917; Capt. Engrs., U. S. A., Oct., 1918; Maj., Engrs., U. S. A., Jan., 1919. Overseas service Mar., 1918–May, 1919. Discharged May 10, 1919. With Army Service Corps at Paris and Neufchateau, France.

STAFFORD, FREDERICK DIAL
Entered service Sept. 3, 1917; Capt., Engrs., N. A., Sept. 27, 1917. Overseas service May 26, 1918–Mar. 1, 1919. Discharged Sept. 4, 1919. Co. Comdr., 105th Engrs.; Engr. Purchasing Office, Paris; Office, Chf. of Engrs., Washington, D. C. Three stars.

STALLINGS, JOHN ROBERT
Sgt., F. A., U. S. A.*

STANFORD, HOMER REED
Entered service May 20, 1898; through all grades in C. E. C., U. S. N., to Capt. at declaration of war. In chg. of all Naval Public Works operations in 1st Naval Dist.

STANLEY, LLOYD LAWRENCE
Entered service Dec. 28, 1917; Lt., Engrs., N. A., Oct. 6, 1917. Discharged Oct. 31, 1919. With 2d Engr. Replacement Regt., Camp Humphreys; in chg. purchase floating equipment for U. S. A. overseas and for rail accessories for War Dept.

STANLEY, WILLIAM EDWARD
Entered service May 14, 1917; 1st Lt. E. O. R. C., June 23, 1917; Capt., Engrs., U. S. A., Apr. 7, 1919. Overseas service Dec. 11, 1917–July 29, 1919. Discharged Aug. 18, 1919. With 101st Engrs.; attached as observer to 7th French Army; student and Instructor, 2d Corps School, A. E. F.; student, Engr. School, Langres, France; with 101st Engrs. during Meuse-Argonne offensive. One star.

STANTON, CHARLES BEECHER
Entered service May 8, 1917; Capt., E. O. R. C., Feb. 16, 1917; Maj., Engrs., N. A., Feb. 13, 1918. Overseas service July 9, 1917–July 17, 1919. Discharged Aug. 5, 1919. With 15th Engrs. as Co. and Bn. Comdr.; Asst. Director, College of Eng., A. E. F. Univ., Beaune, France.

STANTON, WILBOR DICKENS
Capt., Engrs., U. S. A.*

STANTON, WILLIAM LEWIS
Entered service July 13, 1917, as Pvt., Inf., N. A.; 2d Lt., Engrs., N. A., Apr. 1, 1918. Overseas service July 9, 1918–Apr. 22, 1919. Discharged May 7, 1919. With 304th Engrs. in Meuse-Argonne offensive. Two stars.

STARR, FRANK CHARLES
Entered service Feb. 21, 1918, as Capt., Q. M. C., N. A.; Maj., Q. M. C., Constr. Div., U. S. A., July 23, 1919. Const. Q. M., Gen. Hosp. No. 76 and Field Museum Hosp., Chicago; Superv. Const. Q. M., Hosps. and Warehouses, Constr. Div., Washington, D. C.

STARRETT, WILLIAM AIKEN
Entered service May 2, 1917, as Capt., E. O. R. C.; Maj., E. O. R. C., June 13, 1917; Col., Q. M. C., N. A., Mar. 18, 1918. Discharged Mar. 22, 1919. Chairman, Emergency Constr. Comm., War Industries Bd.; Cont. Officer, War Industries Bd., Food Administration and Ordnance Bldgs., War Trade Bd. bldgs.; special investigation of war constr. work for Asst. Secy. of War.

STAYTON, EDWARD MOSES
Entered service Aug. 5, 1917, as Maj., Engrs., N. A.; Lt. Col., Engrs., U. S. A., Sept. 27, 1918. Overseas service May 2, 1918–Apr. 19, 1919. Discharged May 3, 1919. With 110th Engrs. Three stars.

STEARNS, FRED LEROY
Entered service July 15, 1917, as 2d Lt., Inf., N. G.; 2d Lt., Engrs., N. G., May, 1918; 1st Lt., Engrs., U. S. A., Sept. 16, 1918. Overseas service May 17, 1918–Feb. 28, 1919. Discharged Apr. 5, 1919. With 107th Inf. and 102d Engrs., 27th Div. Three stars.

STEARNS, RALPH HAMILTON
Entered service May 4, 1918, as Lt., Jr. Grade, C. E. C., U. S. N. R. F.; Lt., C. E. C., U. S. N. R. F., Feb. 11, 1919. Released from active service July 21, 1919. In Bureau of Yards and Docks, Washington, D. C.; at U. S. Naval Ord. Plant, Charleston, W. Va.

STEEP, JAMES BIGELOW
Entered service Jan. 5, 1918; Capt., Engrs., N. A., Sept. 17, 1917. Discharged June 30, 1919. Supervisor of production, machinery and eng. materials branch, Purchase, Storage and Traffic Div.

STEESE, JAMES GORDON
Entered service June 16, 1902; through all grades in C. of E., U. S. A., to Maj., May 15, 1917; Lt. Col., Aug. 5, 1917; Col., June 19, 1918; Col., G. S., U. S. A., Sept. 12, 1918. Returned to permanent grade of Maj., C. of E., U. S. A. Overseas service, July 13, 1919–Sept. 28, 1919. Organized 1st O. T. C., Ft. Riley; Instr., E. O. T. C., Ft. Leavenworth; with 7th Engrs.; Asst. Chf. of Engrs., U. S. A., in chg. of Personnel, Equipment, Constr. and Maps Div.; Executive Asst., Personnel Branch, and Chf. of Promotions and Assignments Sec., G. S., U. S. A.; special mission overseas as personal representative of Secretary of War. Distinguished Service Medal; Medal of La Solidaridad, 2d Class, Panama. Officer of Order of Prince Danilo I, Montenegro; Croix de Guerre, 2d Class, Greece; Silver Medal for Bravery, Montenegro.

STEEVES, CLARENCE McNAUGHTON
Entered service Dec. 17, 1915, as Lt., Inf., Canadian Army; Capt., Canadian Engrs., May 24, 1918. Overseas service Dec 26, 1916–Mar. 27, 1919. Discharged Aug. 11, 1919. With 115th Bn., Canadian Inf.; Adj., 9th Bn., Canadian Engrs.; Works Officer.

STEINBERG, MAX
1st Lt., C. A. C., U. S. A.*

STEINHAUSER, HARRY HERMAN
Entered service Feb. 20, 1918, as Pvt., 1st Class, Aviation Sec., Sig. C., N. A.; 2d Lt., A. S., N. A., June 12, 1918. Discharged Dec. 12, 1918. Post Maintenance Officer and Engr. Officer in chg. flying field and hangars, Post Field, Ft. Sill; Engr. Officer, 346th Aero Squadron.

STELLHORN, ADOLF
Maj., Engrs., U. S. A., A. E. F.*

STEM, CLIFFORD HOEY
Entered service May 7, 1917; 2d Lt., E. O. R. C., June 21, 1917; 1st Lt., Engrs., N. A., June 3, 1918. Overseas service Dec. 14, 1917–May 6, 1918. Discharged Dec. 6, 1918. With 312th, 101st and 2d Engrs. in U. S. and overseas; detached from A. E. F. to train troops for sapper duty at Camps Lee and Humphreys; 2d Corps Schools, A. E. F.; Asst. Instr., E. O. T. S., Camp Humphreys.

STEPHENS, UEL
Entered service Aug. 24, 1917; 2d Lt., Inf., N. A., Oct. 4, 1917; 1st Lt., Inf., N. A., May 20, 1918; Capt., Inf., U. S. A. Oct. 4, 1918. Discharged Mar. 31, 1919. Guard duty on Mexican border and intensive training in Div. camp.

STERN, EUGENE WASHINGTON
Entered service Jan. 26, 1917, as Maj., E. O. R. C. Overseas service Oct. 27, 1917–Jan. 1, 1919. Discharged Jan. 16, 1919. On staff duty with Chf. of Engrs., Washington, D. C.; road instruction with French 8th Army Sector, Pont à Mousson to Baccarat; Chf. Road Officer, Base Sec. No. 2, A. E. F. One star.

STEVENSON, MARKLEY
Entered service July 24, 1918, as C. Q. M., U. S. N. R. F.; Ensign, U. S. N. R. F., Nov. 15, 1918. Released from active service Jan. 20, 1919. Office, Chf. of Naval Operations, Aviation Div., Washington, D. C.

STEWART, BENJAMIN FRANKLIN, JR.
Entered service Oct. 17, 1918, as 1st Lt., Engrs., U. S. A. Discharged Jan. 3, 1919. Ft. Douglas and E. O. T. S., Camp Humphreys.

STEWART, JOHN
Entered service May 8, 1917; Capt., E. O. R. C., May 16, 1917; Maj., E. O. R. C., Oct. 30, 1917; Lt. Col., Engrs., U. S. A., Aug. 29, 1918. Discharged Oct. 31, 1919. Special duty with Div. Engr., Central Div.; Asst. to Executive Officer and Engr., Inland Waterways Comm., U. S. R. R. Administration; 2d in command 216th and 323d Engrs.; Student Staff School, War Plans Div., Gen. Staff; Asst. to Chf. of Engrs. in transportation matters; special duty, river and harbor work.

STEWART, JOHN TRUESDALE
Entered service May 8, 1917; Maj., E. O. R. C., Apr. 18, 1917; Lt. Col., Engrs., U. S. A., Oct. 18, 1918. Discharged Nov. 13, 1919. Asst. to officer in chg. Equipment Sec., Office, Chf. of Engrs., Washington, D. C.; served on various boards for determining relative qualifications of reserve officers; member, Cont. Review Bd. and Claims Bd., Office, Chf. of Engrs., Washington, D. C., and various other boards.

STEWART, SPENCER JAMES
Capt., Q. M. C., U. S. A.*

STICKLE, HORTON WHITEFIELD
Entered service June 15, 1895; through all grades to Lt. Col., C. of E., U. S. A., at time of retirement Jan. 11, 1916. Recalled to active service as Lt. Col., C. of E., U. S. A., May 21, 1917; Col., Engrs., U. S. A., Aug. 15, 1918. Returned to inactive list Aug. 8, 1919. Engr. Officer, Wheeling, W. Va., and Pittsburgh Dists.; with 216th and 323d Engrs.

STIDHAM, HARRISON
Entered service Oct. 23, 1918, as Cap., Q. M. C., Constr. Div., U. S. A. Discharged Dec. 31, 1918.

STILES, ARTHUR ALVORD
Entered service Apr. 18, 1917, as Maj., E. O. R. C. Resigned Feb. 26, 1918. State reclamation work in Texas.

STILLWELL, HOWARD LOGAN
Entered service Apr. 2, 1918, as Pvt., Depot Brig.; 2d Lt., Q. M. C., Constr. Div., N. A., July 26, 1918; 1st Lt., Q. M. C., Constr. Div., U. S. A., Sept. 11 1918. Discharged Apr. 2, 1919. Asst. to Utility Officer, Camp Gordon, in chg. Water Supply Sec., and later Bldg. and Repair Sec.

STILSON, CHARLES EDWARD
Entered service June 14, 1918, as Ensign, C. E. C., U. S. N. R. F.; Lt., Jr. Grade, C. E. C., U. S. N. R. F., Apr. 1, 1919. Released from active service Nov. 30, 1919. In chg. constr., 3d Naval Dist.

STINEMAN, NORMAN MERRITT
Entered service Sept. 2, 1917, as Capt., Engr. R. C. Overseas service May 10, 1918–July 13, 1919. Discharged July 29, 1919. With 33d Engrs.; in chg. car repair yard, Nevers, France.

STIVERS, ARTHUR DUCAT
Entered service Apr. 21, 1917; Capt., Inf., N. A., Aug. 15, 1917; Maj., Inf., U. S. A., Feb. 24, 1919. Overseas service Dec. 20, 1917–June 20, 1919. Discharged July 24, 1919. Asst. Chf. of Staff, 40th Div.; Observer and Liaison Officer with 6th British Div.; duty with 26th Inf., 2d Brig., H. Q., and Div. H. Q., 1st Div., A. E. F.; Gen. Staff, 6th Army Corps, A. E. F.; Asst. Chf. of Staff, 4th Army Corps, A. E. F. Regtl. and Corps citations. Four stars. One wound.

STOCKER, LESLIE WRIGHTSON
Entered service Oct. 23, 1918, as 1st Lt., Engrs., U. S. A. Discharged Feb. 4, 1919. With 403d Engrs.

STOREY, FRANKLIN STEVENS
Entered service May 15, 1917; 1st Lt., Engrs., N. A., Aug. 15, 1917; Capt., Engrs., U. S. A., Oct. 25, 1919. Overseas service Sept. 14, 1917–June 18, 1919. Discharged June 18, 1919. With Sec. Engr., Advance Sec., A. E. F., constr. hosps., camps, etc.

STOWE, HENRY DANIELS
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 10, 1917. Overseas service Jan. 3, 1918–May 26, 1919. Discharged June 26, 1919. With T. C.; 17th Engrs. at Brest and St. Nazaire, 116th Engrs. at Angers, and 307th Engrs. in Meuse-Argonne offensive. One star.

STRACHAN, JOSEPH JOSLIN
Entered service June 26, 1917, as Lt., Jr. Grade, C. E. C., U. S. N. R. F.; Lt., C. E. C., U. S. N. R. F., Oct. 15, 1917. Released from active service Oct. 21, 1919. Duty at Naval Academy; Acting Public Works Officer, Boston Navy Yard; Sr. Asst. to Public Works Aide on staff of Commandant, 1st Naval Dist.

STRACHAN, NORMAN FRASER
Pvt., Engrs., U. S. A.*

STRECKER, ROBERT AUGUST
Entered service May 10, 1917; Capt., Engrs., N. A., Aug. 10, 1917; Maj., Engrs., N. A., Aug. 1, 1918. Overseas service Sept. 6 1918–July 26, 1919. Discharged Aug. 7, 1919. Co. and Bn. Comdr., 309th Engrs.; C. O., U. S. Troops at Savenay, France.

STREET, JOHN ZADOK
Entered service Sept. 19, 1917, as Pvt., Engrs., N. A.; M. E., Sr. Grade, Engrs., U. S. A. Overseas service Feb. 27, 1918–Mar. 12, 1919. Discharged Mar. 22, 1919. With 308th Engrs.; with 26th Engrs. in 2d Army; Asst. to Engr. in chg. constr., Langres, France.

STREHAN, GEORGE ERNEST
Entered service Aug. 25, 1917; 2d Lt., F. A., N. A., Dec. 15, 1917; 1st Lt., F. A., U. S. A., Sept. 5, 1918. Overseas service June 16, 1918–Feb. 22, 1919. Discharged Mar. 2, 1919. With 351st F. A. as Orientation Officer and Battery Comdr.

STRICKLER, GRATZ BROWN
Entered service July, 1917, as Maj., E. O. R. C.; Lt. Col., Q. M. C., Constr. Div., U. S. A., Sept., 1918. Discharged Feb., 1919. Const. Q. M., Camp Hancock and Baltimore Depot Warehouses.

STRICKLER, THOMAS JOHNSON
Entered service Sept. 25, 1917; Capt., Engrs., N. A., Sept. 7, 1917; Maj., T. C., U. S. A., Oct. 12, 1918. Overseas service Jan. 7, 1918–June 8, 1919. Discharged June 12, 1919. With 114th and 116th Engrs.; C. O., R. T. C. Schools, and C. O., Camp de Grasse, Tours, France.

STRINGFELLOW, HORACE
Entered service Aug. 9, 1918, as Capt., Engrs., N. A. Discharged Dec. 12, 1918. E. O. T. S., Camp Humphreys; with 150th Engrs.

STRONACH, ROBERT SUMMERS
Lt., Canadian Engrs., B. E. F.*

STUART, EDWARD
Entered service Aug. 15, 1918, as Capt., San. C., U. S. A.; Maj., San. C., U. S. A., Feb. 17, 1919. Overseas service Sept. 15, 1918–May 6, 1919. Discharged May 10, 1919. Special duty at Salonika front as San. Adviser to Serbian Army and Govt. Commander, Order of St. Sava, Serbia.

STUPP, JOHN GEORGE
Entered service Sept. 1, 1917; 1st Lt., Ord. C., N. A., Nov. 27, 1917; 1st Lt., C. W. S., N. A., July 1, 1918. Discharged Dec. 27, 1918. Edgewood Arsenal, as Asst. in chg. design of bldgs. for poison gas plant and shell filling plant.

STURTEVANT, CARLETON WILLIAM
Entered service May 8, 1917; Maj., E. O. R. C., Feb. 23, 1917; Lt. Col., E. O. R. C., July 6, 1917; Col., Engrs., U. S. A., Nov. 13, 1918. Overseas service July 9, 1917–Apr. 27, 1919. Discharged May 7, 1919. With 15th Engrs. as Bn. Comdr., 2d in command, and C. O. Citation from Gen. Pershing for meritorious service.

SULLIVAN, JOHN FRANCIS
Entered service Mar. 3, 1918; Capt., E. O. R. C., June 13, 1917; Maj., Q. M. C., Constr. Div., U. S. A., Aug. 24, 1918. Discharged July 20, 1920. Asst. to Director of Constr., U. S. Explosive Plant *C*, Nitro, W. Va.; in chg. constr., Nitrate Plant No. 2, Muscle Shoals, Ala.

SUMMERS, RICHARD ELVIN JEWELL
Entered service May 25, 1917; 1st Lt., E. O. R. C., June 7, 1917; Capt., Engrs., N. A., July 30, 1918. Overseas service July 9, 1917–Sept. 9, 1918. Discharged Mar. 29, 1919. With 15th Engrs., constr. work at Gievres Intermediate Storage Depot, France, and elsewhere.

SUTER, RUSSELL
Entered service May 16, 1917; Capt., E. O. R. C., June 11, 1917. Overseas service Nov. 26, 1917–July 4, 1919. Discharged July 14, 1919. With 114th Engrs.; Water Supply Sec., H. Q. Div. of Constr. and Forestry, S. O. S., A. E. F.; with Sec. Engr., Intermediate Sec. East, A. E. F.

SUTTON, FRANK
Entered service June 18, 1917; Maj., E. O. R. C., Jan. 23, 1917. Overseas service Aug. 14, 1918–Apr. 2, 1919. Discharged Apr. 7, 1919. Bn. Comdr., 25th Engrs., Camp Devens; organized 29th Engrs., Camp Devens; organized 2d Bn., 604th Engrs., Washington Barracks; Map Sec., Office, Chf. Engr., A. E. F.

SWAREN, JOHN WILLIAM
Entered service May 8, 1917, as Capt., E. O. R. C.; Maj., Engrs., N. A., July 30, 1918. Overseas service May, 1918–Sept. 12, 1918. Discharged May 7, 1919. Instr. E. O. T. S.; Supply Officer, 318th Engrs.

SWARTZ, FREDERICK PETER
Entered service Aug. 26, 1918, as Capt., Engrs., U. S. A. Discharged Jan. 1, 1919. Co. Comdr., 70th Engrs.

SWARTZ, LEON
Entered service June 2, 1917, as M. E., Jr. Grade, C. of E., U. S. A.; M. E., Sr. Grade C. of E., U. S. A., Mar. 1, 1918; 2d Lt., Engrs., U. S. A., Oct. 20, 1918; 1st Lt., Engrs., U. S. A., Apr. 9, 1919. Overseas service July 9, 1917–Apr. 27, 1919. Discharged May 12, 1919. With 15th Engrs., principally ry. constr.

SWEENEY, HARRY CLINTON
Entered service June 29, 1917; Capt., Q. M. C., N. A., May 14, 1917; Maj., Q. M. C., Constr. Div., U. S. A., Oct. 19, 1918. Discharged Mar. 12, 1920. Asst. to Const. Q. M., Camp Devens; Const. Q. M., War Telephone and Telegraph Bldgs. for War and Navy Depts., Washington, D. C., and at Camp Meade, Camp Doniphan and Ft. Wood; Supervising Const. Q. M. for Panama Canal Zone, Philippines and Mexican Border projects, Washington, D. C.

SWEETSER, CHARLES HERBERT
Entered service Jan. 17, 1918; Capt., Engrs., N. A., Jan. 3, 1918. Overseas service May 22, 1918–June 29, 1919. Discharged July 3, 1919. Co. Comdr., 41st and 43d Engrs.; on

various forestry projects in A. E. F.; Wood Supply Officer, Liffol-le-Grand, France; Supt. Roads, Base Sec. No. 1, A. E. F.

SWEETSER, ERNEST OSGOOD
Entered service Sept. 20, 1917; Capt., Engr. R. C., Aug. 21, 1917. Overseas service Jan. 27, 1918–Feb. 23, 1919. With 104th Engrs. at Camp McClellan; with 116th Engrs. in A. E. F. as Classification and Replacement Officer.

SWETT, EVERETT HAROLD
Entered service July 31, 1918; Capt., Engr. R. C., Jan. 28, 1918. Discharged Dec. 18, 1918. Training camp at Camp Humphreys; with various training regts. as Co. Comdr., Personnel Officer, Exchange Officer, Camp Humphreys.

SYKES, GEORGE†
Maj., Engrs., U. S. A., A. E. F.*

SYMONDS, GEORGE ROSCOE BLAINE
Entered service Feb. 11, 1918; Capt., E. O. R. C., June 13, 1917. Overseas service Sept. 30, 1918–June 18, 1919. Discharged Aug. 5, 1919. Co. Comdr., Bn. Adj., and Regtl. Adj., 605th Engrs.

TAINTER, FRANK STONE
Entered service Sept. 4, 1917; Maj., Engrs., N. A., Oct., 1917; Lt. Col., Engrs., U. S. A., Aug. 16, 1919. Overseas service Oct. 1, 1917–Nov. 30, 1917. Released from active service Sept. 11, 1919. In office Adj. Gen. of the Army.

TALBOTT, KENNETH HAMMET
Entered service Apr. 18, 1918, as 1st Lt., Q. M. C., Constr. Div., N. A.; Capt., Q. M. C., Constr. Div., U. S. A., Aug. 24, 1918. Discharged Jan. 17, 1919. Asst. to Const. Q. M., Army Reserve Depot, New Cumberland, Pa.

TANDROW, WALTER STEPHEN
Entered service Oct. 11, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 24, 1918. With 55th Engrs.

TATE, ROBERT L'HOMMEDIEU
Entered service May 5, 1917; 1st Lt., E. O. R. C., Apr. 16, 1917; Capt., Engr. R. C., July 12, 1918. Overseas service May 8, 1918–June 19, 1919. Discharged June 26, 1919. Co. Comdr., Bn. Adj., and Regtl. Personnel Adj., 303d Engrs; St. Mihiel and Meuse-Argonne offensives. Two stars.

TAUSSIG, JOHN WRIGHT
Entered service Oct. 15, 1918, as Pvt., Engrs., U. S. A. Discharged Nov. 27, 1918. E. O. T. S., Camp Humphreys.

TAY, SAMUEL WRIGHT
Entered service Apr. 7, 1917; Lt., N. N. V., Oct. 12, 1916. Overseas service June, 1917–Jan., 1919. Released from active service Apr. 15, 1919. U. S. S. *St. Louis* and U. S. S. *Wenonah*, escort duty on Atlantic Ocean and Mediterranean Sea.

TAYLOR, ARTHUR
Entered service June 13, 1917; 1st Lt., Engrs., N. A., Mar. 5, 1918; Capt., Engrs., U. S. A., Sept. 5, 1918. Overseas service Mar. 28, 1918–Sept. 17, 1918. Discharged Nov. 29, 1918. With 23d Engrs. in road work in A. E. F.; returned to U. S. to train and command sapper troops.

TAYLOR, EDWIN ALEXANDER
Entered service June 28, 1917, as Maj., E. O. R. C. Overseas service July 10, 1918–June 6, 1919. Discharged July 1, 1919. C. O., 526th and 508th Engrs.; in chg. constr. in Adv. Sec., S. O. S., A. E. F. One star.

TAYLOR, EDWY LYCURGUS
Entered service July 9, 1918; 1st Lt., Engr. R. C., July 2, 1918. Overseas service July 10, 1918–June 26, 1919. Discharged July 2, 1919. With 46th Engrs. (later 46th and 30th Cos., T. C.), in ry. maintenance work; Co. Comdr.

TAYLOR, HAROLD ALEXANDER
Entered service Mar. 6, 1918, as Pvt., Medical C., N. A.; Sgt., Medical C., U. S. A., Sept., 1918; Sgt., 1st Class, Medical C., U. S. A., Oct., 1918. Overseas service July 4, 1918–Apr. 28, 1919. Discharged May 7, 1919. Office, Personnel Adj., Base Hosp. No. 48.

TAYLOR, HARRY
At declaration of war Col., C. of E., U. S. A.; Brig. Gen., U. S. A., Aug. 5, 1917. Overseas service May 28, 1917–Sept. 20, 1918. Chf. Engr. Officer, A. E. F.; Asst. to Chf. Engr. Officer, A. E. F.; Asst. to Chf. of Engrs., Washington, D. C. Distinguished Service Medal; Commandeur, Legion d'Honneur.

TAYLOR, HENRY
Entered service May 1, 1917; 1st Lt., E. O. R. C., Feb. 12, 1917; Capt., E. O. R. C., Aug. 1, 1917; Maj., Engr. R. C., Apr. 1, 1918. Overseas service June 29, 1918–July 23, 1919. Discharged Aug. 15, 1919. Co. and Bn. Comdr., 304th Engrs.; Gen. Staff College, A. E. F.; Adj., 58th Inf. Brig.; with G-2, Gen. Staff, 3d Army; Meuse-Argonne offensive. Two stars.

TAYLOR, HUGH McGEHEE
Entered service Feb. 27, 1918, as Maj., T. C.; Lt. Col., T. C., U. S. A., Apr. 2, 1919. Overseas service Mar. 14, 1918–Sept. 5, 1919. Asst. to Gen. Mgr., T. C., A. E. F.; Gen. Supt., 16th Grand Div., T. C., A. E. F. Citation from Gen. Pershing for meritorious service; Officier d'Academie.

† Died Aug. 21, 1919.

TAYLOR, NELSON
Entered service Apr. 6, 1917; Lt., N. N. V., Apr. 18, 1917; Lt. Comdr., U. S. N. R. F., Sept. 21, 1918. Overseas service June 15, 1917–Nov. 11, 1919. Released from active service Jan. 30, 1920. Watch and Div. Officer, U. S. S. *St. Louis*; Navigator, U. S. S. *Bridgeport*. One star.

TAYLOR, PRESLEY MORGAN
Entered service Oct., 1916, British Army; 2d Lt., Royal Engrs., Jan., 1917; 1st Lt., Royal Engrs., June, 1918. Overseas service Mar., 1917–Feb., 1919. Discharged Feb., 1919. With B. E. F. in France, Belgium, Italy and Germany. Croix de Guerre, Belgium.

TAYLOR, WILLIAM THOMAS
Entered service Sept., 1915, as Capt., Loyal North Lancers, British Army; Staff Capt., Royal Air Force, June, 1918. Overseas service July 1916–Jan. 1918. Discharged Apr. 1919. Co. Comdr.; Instr., trench warfare; Equipment Officer and Staff Capt., Royal Flying Corps.

TAYLOR, WYLLYS HARD
Entered service May 8, 1917; Capt., E. O. R. C., June 19, 1917. Overseas service July 30, 1918–July 25, 1919. Discharged July 26, 1919. Co. Comdr., 306th Engrs.; Polish Convoy Officer; Meuse-Argonne offensive. One star.

TEN HAGEN, HENRY
Entered service May 15, 1917; 2d Lt., Engr. R. C., Jan. 1, 1918; 1st Lt., Engrs., U. S. A., Aug. 31, 1918. Overseas service Dec. 11, 1917–Aug. 20, 1918. Discharged July 11,1919. With 303d Engrs., Camp Dix; with 2d Engrs.; training engr. troops, Camp Forrest; with 3d Engrs., Panama.

TENNEY, WILLIS ROBINSON
Entered service May 14, 1917; Capt., Engrs., N. A., Aug. 15, 1917. Overseas service Dec. 24, 1917–May 31, 1919. Discharged June 6, 1919. With 304th and 317th Engrs.; Depot Sec., Office, Chf. Engr., A. E. F., in chg. depots at Is-sur-Tille, Brest and Pontarlier, France.

THOMAS, CHARLES DURA
Entered service May 15, 1917; Capt., E. O. R. C., Apr. 16, 1917. Overseas service Feb. 18, 1918–July 5, 1919. Discharged July 29, 1919. Office, Const. Q. M., Washington, D. C.; with 507th Engrs.; in chg. bldg. constr. at American Hosp., Beaune, under Div. of Constr. and Forestry, S. O. S., A. E. F. One star.

THOMAS, RALEIGH COLSTON
Entered service Sept., 1917; Capt., Engrs., N. A., Dec. 1917; Maj., Engrs., U. S. A., Aug., 1918. Discharged Dec. 31, 1918. Co. Comdr., 530th Engrs., Camp Funston; Co. Comdr., 49th Engrs., Ft. Myer; C. O., 76th Engrs., Camp Leach.

THOMAS, WILLIAM EDWARD
Entered service Jan., 1918, as Capt., Q. M. C., N. A. Discharged Dec. 19, 1918. Asst. Const. Q. M., Camp Sevier and Camp Benning.

THOMAS, WILLIAM MICHAEL
Entered service June 10, 1918, as Capt., Engrs., N. A. Discharged Jan. 10, 1919. Camps Lee and Humphreys, with 7th and 4th Engr. Training Regts.

THOMPSON, JAMES ARTHUR
Entered service June 12, 1918, as Pvt., C. A. C., N. A.; 2d Lt., C. A. C., U. S. A., Sept., 25, 1918. Discharged Dec. 4, 1918. Student and Instr., C. A. C. School, Ft. Monroe; student, Anti-Aircraft School, Ft. Monroe.

THOMPSON, SANFORD ELEAZER
Entered service Dec. 19, 1917, as Maj., Ord. R. C.; Lt. Col., Ord. Dept., U. S. A., Oct. 14, 1918. Discharged Dec. 21, 1918. Chf. of Progress Sec., Office, Chf. of Ord., Washington, D. C.

THOMPSON, WILFORD ASHFORD
Entered service Sept. 11, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 27, 1918. Camp Humphreys, with 4th Engr. Training Regt.

THOMSEN, SAMUEL LOCKE
Entered service Dec. 28, 1917; Capt., E. O. R. C., Sept. 25, 1917. Overseas service June 29, 1918–June 1, 1919. Discharged June 21, 1919. With 57th Engrs. in U. S. and France; Supt., Inland Water Transport, Le Havre to Paris; C. O., Camp 3, Annex, Base Sec. No. 1, A. E. F.

THOMSON, FRED MORTON
Entered service Feb. 18, 1918, as Capt., Engrs., N. A. Overseas service June 6, 1918–Apr. 4, 1919. Discharged Apr. 19, 1919. Co. Comdr., 39th Engrs.; special duty in ry. operation, A. E. F.

THORNTON, LOUIS EARLE
Entered service June 22, 1918, as Lt., C. E. C., U. S. N. R. F. Overseas service July 19, 1918–Mar. 15, 1919. Aide for Public Works at Naval Sta., Pauillac, France; Public Works Officer, Naval Air Stas., Moutchic and St. Croisic, France.

THROOP, GEORGE HUNTINGTON
Entered service May 14, 1917; Capt., Engrs., N. A., July 21, 1917; Maj., Engrs., U. S. A., Aug. 24, 1918; Lt. Col., Engrs., U. S. A., Feb. 24, 1919. Overseas service Feb. 18, 1918–May 30, 1919. Discharged June 19, 1919. Executive Officer, Is-sur-Tille Engr. Depot; C. O., 1st Army Engr. Depot, Leonval, France; Engr. Supply Officer, 2d Army; with 24th Engrs., St. Mihiel, Meuse-Argonne and Toul Sector operations. Three stars.

THURBER, CLINTON DRAPER
Entered service Jan., 1904; through all grades in C. E. C., U. S. N., to Comdr., July 1, 1918. With Bureau of Yards and Docks, Navy Dept., Washington, D. C.

THURSTON, EUGENE TRUE
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 6, 1917. Overseas service Feb. 28, 1918–Feb. 28, 1919. Discharged Aug. 18, 1919. With 25th Engrs. at Camp Devens and in France; constr. work at Montoir, France; Intelligence Sec., Gen. Staff, G. H. Q., A. E. F.; Asst. to Port Utilities Officer, New York.

TILDEN, CHARLES JOSEPH
Entered service May 8, 1917; Capt., E. O. R. C., Feb. 23, 1917. Released from active service May 31, 1917. Ft. Myer, Va.

TINKHAM, RALPH RUSSELL
Entered service Sept. 7, 1918, as Capt., Engrs., U. S. A. Discharged Mar. 10, 1919. With 420th Engrs., Camp Lewis; E. O. T. S., Camp Humphreys; Office, Chf. of Engrs., Washington, D. C.

TINSLEY, ROBERT BRUCE
Entered service May 1, 1918; Capt., Engrs., N. A., Jan. 26, 1918. Overseas service Sept. 30, 1918–June 18, 1919. Released from active service July 11, 1919. Co. Comdr., 605th Engrs.; Staff Corps, 1st Army, A. E. F.

TIRRELL, CHARLES EDWARDS
Entered service Mar. 9, 1918, as cadet, A. S., N. A.; 2d Lt., A. S., N. A., Apr. 27, 1918. Overseas service July 6, 1918–June 19, 1919. Discharged June 23 1919. Executive Officer, Airplane Div., Technical Sec., A. S., H. Q., Paris.

TODD, FRANK HERBERT
Entered service Oct. 26, 1917, as Maj., Q. M. C., N. A. Discharged Mar. 29, 1919. Officer in Chg. Utilities, Camp Travis; Camp Bowie; Officer in chg. constr. and Utilities Officer, Camp Las Casas, Porto Rico.

TODD, OLIVER JULIAN
Entered service Sept. 2, 1917; Capt., E. O. R. C., July, 1917. Overseas service Jan. 4, 1918–July 5, 1919. Discharged July 10, 1919. Bn. Adj., 20th Engrs., Washington, D. C.; Chf. Asst. Engr. in constr. Mars hosp., France; Engr. in chg. constr. Mesves hosp., France; Eng. Agent Disbursing Officer, Nevers, France; Road supervisory work. Diploma from Gen. Pershing for meritorious service.

TOLLES, FRANK CLIFTON
Entered service Apr. 30, 1917, as Pvt., N. G.; Cpl., Engrs., N. G., July 15, 1917; Sgt. Engrs., N. G., Aug. 1, 1917; Sgt., 1st Class, Engrs., N. G., Sept. 1, 1917; 1st Lt., Engrs., N. G., Nov. 15, 1917; Capt., Engrs., U. S. A., Nov. 13, 1918. Overseas service June 23, 1918–Apr. 1, 1919. Discharged Apr. 16, 1919. With 112th Engrs.; Topographic Officer 112th Engrs. Croix de Guerre; Divisional citation. Three stars.

TOMLINSON, ALFRED THOMAS
Entered service July, 1915, as Capt., Canadian Engrs.; Maj., Canadian Engrs., Dec. 6, 1915. Released from active service Apr. 1, 1919. Insp., small arms ammunition, Canadian Army; special assignment from London War Office to Guatemala, Aug. to Oct., 1915.

TOMPKINS, ROBERT HARRY
Entered service Apr. 1, 1917, as 2d Lt., Inf., Tex. N. G.; 1st Lt., Engrs., Tex. N. G., Aug. 5, 1917; 1st Lt. Engrs., N. A., Nov. 1, 1917. Overseas service July 16, 1918–June 1, 1919. Discharged July 9, 1919. With Const. Q. M., Camp Bowie; with 111th Engrs. in St. Mihiel and Meuse-Argonne offensives; highway maintenance and sawmill operation, 16th Training Area, A. E. F. Two stars.

TOPPING, PERRY
Entered service May 10, 1917, as Capt., E. O. R. C. Discharged Mar. 15, 1919. Overseas service July 20, 1918–Dec. 25, 1918. With 5th Engrs. in U. S. and France. Wounded once.

TORRANCE, WILLIAM MARTIN†
Capt., Engrs., U. S. A.*

TOWNSEND, CURTIS McDONALD
Entered service June 15, 1875; through all grades in C. of E., U. S. A., to Col. at declaration of war. Overseas service July 26, 1917–Nov. 5, 1918. Retired Feb. 15, 1920. C. O., 12th Engrs. with British 3d and 5th Armies; Engr. Purchasing Officer, Paris. Officier, Legion d'Honneur.

TOYNE, JOHN WILSON
Entered service Aug. 9, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Mar. 22, 1919. Asst. to Utilities Officer, Camps Custer and Meade; Utilities Officer, Chickamauga Park, Ga.

TRACY, CLARENCE CURTIS
Entered service Aug. 15, 1917; Capt., C. A. C., N. A., Nov. 15, 1917. Overseas service Aug. 9, 1918–Feb. 16, 1919. Discharged Mar. 5, 1919. Battery Comdr., 68th Arty., C. A. C.

TRACY, HERBERT HERMAN
Entered service Dec. 28, 1917; 1st Lt., E. O. R. C., Sept. 25, 1917. Overseas service Mar. 27, 1918–June 11, 1919. Discharged June 18, 1919. With 23d Engrs., road work in Adv. Sec., A. E. F., and as army troops with 1st Army in St. Mihiel and Meuse-Argonne offensives. Two stars.

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† Died May 18, 1920.

TRAVERS-EWELL, ANDREW
Entered service Dec., 1914, American Ambulance Service; Sgt., American Ambulance Service, Apr., 1915; Capt., San Div., French Army, Aug., 1915. One year overseas service. Discharged Dec., 1915. All service in French Army.

TRESER, ALBERT PAUL
Entered service June 7, 1918, as Pvt., Engrs., N. A.; Cpl., Engrs., N. A., July 18, 1918; Sgt., Engrs., U. S. A., Dec. 10, 1918. Overseas service Sept. 30, 1918–June 18 1919. Discharged June 23, 1919. With 472d and 605th Engrs.

TRIMPI, ALLAN LITTELL
Entered service Apr. 18, 1917; 2d Lt., Engrs., N. G. N. J., Apr. 24, 1917; 1st Lt., N. G. N. J., May 10, 1917. Discharged Jan. 8, 1919. Topographical surveys, Camp Dix, Camp Lee, Camp McClellan; Asst. to Const. Q. M., Camp Lee; Topographical Officer, 104th Engrs.; transferred to Ord. Dept., on design and operation of shell-loading plants; assisted in organization of company of 104th Engrs.

TROUT, ALEXANDER LINN
Entered service Oct., 1917, as Capt., Engrs., N. A. Overseas service Jan., 1918–Feb., 1919. Discharged Feb. 12, 1919. Technical Information Sec., Office Chf. Engr., A. E. F.; Personnel Officer, Div. Military Eng. and Engr. Supplies, S. O. S., A. E. F.

TROWBRIDGE, ALFRED LOCKWOOD
Entered service Oct. 24, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 24, 1918. Fort Douglas and Camp Humphreys.

TRUE, ALBERT OTIS
Entered service Sept. 26, 1917, as Capt., Engrs., N. A. Office, Chf. of Engrs., Washington, D. C.; Officer in chg. Water Supply School, Camp Humphreys; with 2d Engr. Training Regt.; with 78th Engrs. as Supply Officer and 2d in command; Officer in chg. Big Bethel Water Development; Utilities Officer, Hosp. No. 43 and Newport News, Va.

TRUEBLOOD, PAUL McGEORGE
Entered service Oct. 6, 1917, as Lt., U. S. N. R. F. Overseas service Nov. 3, 1917–Dec. 16, 1917, and Jan. 6, 1918–Dec. 26, 1918. Released from active service Feb. 25, 1919. Executive Officer, U. S. S. *William Rockefeller;* Navigator, U. S. S. *Aroostook* minelayer in North Sea.

TURLEY, OMNER JAY
Entered service Apr. 26, 1917; Capt., E. O. R. C., Apr. 22, 1917. Overseas service Nov. 19, 1917–Nov. 29, 1918. Discharged June 20, 1919. Asst. Mustering Officer, Camp Lewis; with British forces; classification of engr. officers, A. E. F.; special work at Hendaye, France, to open traffic between Spain and A. E. F.; Zone Major and Liaison Officer.

TURNER, DANIEL NORMAN
Entered service May 7, 1917; 2d Lt., E. O. R. C., June 11, 1917; 1st Lt., Engrs., N. A., Apr. 1, 1918. Overseas service July 8, 1918–May 29, 1919. Discharged June 26, 1919. With 304th Engrs. as Divisional and Corps engrs. with 1st Army in St. Mihiel and Meuse-Argonne offensives; War Damage Bd., American Peace Comm. Two stars.

TURNER, HOWARD MOORE
Entered service July 18, 1918; 1st Lt., Engr. R. C., Aug. 3, 1918. Discharged Dec. 26, 1918. E. O. T. S., Camp Lee; with 606th Engrs., Camp Humphreys.

TURNER, NATHANIEL PARKER
Entered service May, 1917; Capt., E. O. R. C., Apr. 16, 1917; Maj., Engrs., U. S. A., Sept. 4, 1918. Overseas service July 16, 1918–June 1, 1919. Discharged July 5, 1919. With 111th Engrs. as Divisional and Corps engrs. with 1st Army in St. Mihiel and Meuse-Argonne offensives; War Damage Bd., American Peace Comm. Two stars.

TUSKA, GUSTAVE ROBITSCHER
Entered service Sept. 24, 1917, as Maj., E. O. R. C. Discharged July 28, 1919. In training Camps Lee and Humphreys; on staff, Chf. of Engrs., Washington; Member Steel Committee, War Industries Bd.; advisory capacity to Plant Facilities Sec., Div. of Purchase, Storage and Traffic.

TYLER, RICHARD GAINES
Entered service Aug. 1, 1918, as Capt., Q. M. C., N. A. Discharged Mar. 5, 1919. Officer in chg. water, sewers, plumbing and eng., Camp Lee.

TYSON, WILLIAM CLAUDE
Entered service Sept. 25, 1917, as Capt., Engrs., N. A. Overseas service Feb. 27, 1918–Jan. 2, 1919. Discharged Mar. 13, 1919. C. O., Camp Custer troops; at H. Q., Paris.

UHLER, WILLIAM DAVID
Entered service Jan. 24, 1918, as Maj., Q. M. C., N. A.; Lt. Col., M. T. C., N. A., June 5, 1918. Discharged Nov. 23, 1918. Chf. of Operations Div., M. T. C.; member Federal Highway Council, representing War Dept., detailed as asst. to Maj. Gen. Goethals.

VAN AMBURGH, THOMAS ALBERT
Entered service Sept. 11, 1918, as 1st Lt., Q. M. C., Constr. Div., U. S. A. Discharged July 16, 1919. Asst. to Utilities Officer, Camp Beauregard; Asst. to Const. Q. M., Camp Benning.

VAN BUREN, MAURICE PELHAM
Entered service Oct. 16, 1917, as 2d Lt., Engrs., U. S. A.; 1st Lt., Engrs., U. S. A., Feb. 22, 1918; Capt., Engrs., U. S. A., Apr. 6, 1919. Overseas service July 9, 1918–June 6, 1919. Discharged Aug. 8, 1919. With 310th Engrs. at Camp Custer; with 602d Engrs. at Camp

Devens and in A. E. F.; Asst. Chf. Engr., 6th Army, 3d Army and 9th Army Corps; Asst. Camp Engr., Camp Benning; Marbache Sector. One star.

VANDEMOER, JOHN JAY
Entered service May 25, 1918; Capt., Engrs., N. A., May 15, 1918. Overseas service Sept. 29, 1918–Mar. 7, 1919. Discharged Mar. 28, 1919. Organized, and C. O., 467th Engrs. One star.

VANDEMOER, NICHOLAS CORNEILIUS
Entered service Dec. 28, 1917; Capt., Engrs., N. A., May 1, 1918. Overseas service June 29, 1918–Sept. 26, 1919. Discharged Oct. 16, 1919. With 529th Engrs.; Supt., Water Supply and Sewerage, Mesves Hosp., France; Supt., Pontanezan Ry. Camp.

VANDERHOOF, ARNOLD HINES
Ordered to active duty from retired list Apr. 9, 1917, as Ensign, U. S. N.; Lt., U. S. N., July 1, 1918. Placed on inactive list May 1, 1919. In chg. high-power radio station, New Brunswick, N. J.; in chg. maintenance high-powered radio stations, Bureau, Steam Eng., Navy Dept.

VANDERVOORT, BENJAMIN FRANKLIN
Entered service Jan. 5, 1918; Capt., E. O. R. C., Sept. 7, 1917. Asst. to Chf. of Constr. Div.; Asst. to Const. Q. M., Chemical Plant No. 4, Saltville, Va.; Const. Q. M., Portsmouth Water Development, Suffolk, Va., and South Charleston Transmission Lines, Charleston, W. Va.

VANDEVANTER, ELLIOTT
Capt., Engrs., U. S. A., A. E. F.*

VAN ETTEN, PERCY HIXON
Entered service Dec. 28, 1917; 1st Lt., Engrs., N. A., July 30, 1917. Overseas service Mar. 30, 1918–July 9, 1919. Discharged Aug. 4, 1919. With 23d Engrs. in convoy work; Transportation Officer on road work during Meuse-Argonne offensive. One star.

VAN NESS, RUSSELL ALGER
Entered service Apr. 16, 1917, as 2d Lt., E. O. R. C.; 1st Lt., Engr. R. C., Dec. 14, 1917; Capt., Engrs., U. S. A., Aug. 24, 1918. Overseas service Aug. 31, 1918–June 22, 1919. Discharged July 17, 1919. Instr., 2d and 3d training camps; Co. Comdr. and Adj., 604th Engrs. with 5th Corps, constr. roads and light rys. One star.

VAN ORNUM, SAMUEL JUDSON
Entered service Oct., 1918, as Capt., Engrs., U. S. A. Discharged Aug., 1919. E. O. T. S., Camp Humphreys; report on water supply for new post, Camp Humphreys; Instr., Camp Humphreys.

VAN PELT, SUTTON
Entered service Sept. 1, 1917; Capt., E. O. R. C., June 19, 1917. Discharged June, 1918. With 110th Engrs. at Camp Doniphan with 27th Engrs., Camp Meade.

VANSITTART, GEORGE EDWARD
Entered service 1914 as Subaltern, F. A., Canadian Army; Capt., F. A., Canadian Army, 1915; Maj., F. A., Canadian Army, 1916. With 14th Battery; overseas service with 13th Battery, 4th Canadian F. A. Brigade. Mentioned in dispatch by Gen. Sir Douglas Haig for gallant and distinguished service in the field. Died May 14, 1916, from wounds received in action in France.

VAN SUETENDAEL, ACHILLE OCTAVE
Entered service Sept. 2, 1917; Capt., E. O. R. C., June 28, 1917. Discharged Oct. 28, 1919. General Engr. Depot, Washington, D. C.; Div. of Purchases, Storage and Traffic, Gen. Staff.

VAN ZILE, HARRY LEE
Entered service Sept. 13, 1917; Maj., Engrs., N. A., July 20, 1917. Overseas service Sept. 23, 1917–Jan. 29, 1919. Discharged Jan. 31, 1919. Chf. of Plant Constr. Sec., Div. of Constr. and Forestry, S. O. S., A. E. F. Citation from Gen. Pershing for meritorious service; Officier d'Academie, with silver Palms.

VAUGHAN, HENRY FRIEZE
Entered service Dec. 12, 1917, as Capt., San. C., N. A. Discharged Jan. 28th, 1919. Camp Epidemiologist and San. Officer, Camp Upton; member, Pneumonic Comm. appointed by the Surgeon Gen.; Camp Wheeler and Washington, D. C.

VERNON, STEPHEN BARKER
Entered service Aug. 13, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 29, 1918. With 9th Engr. Training Regt.

VERRILL, GEORGE ELLIOT
Entered service Oct. 17, 1917; Maj., E. O. R. C., Feb. 16, 1917. Discharged Dec. 6, 1918. Dist., Engr., Yellowstone Natl. Park; E. O. T. S., Camp Humphreys; various short assignments.

VINCENT, JAMES IRVING
Capt., Ord., U. S. A.*

VINCENT, WILLIAM HUBERT
1st Lt., Engrs., U. S. A.*

VOGDES, JOSEPH JOHNSON
Entered service Jan. 21, 1918, as Capt., Engrs., N. A. Overseas service Feb. 10, 1918–Jan. 13, 1919. Discharged Jan. 17, 1919. Asst. Engr. of Ports, Engr. of Constr., T. C., A. E. F.

VOGLESON, JOHN ALBERT
Entered service Jan. 27, 1918; Maj., San. C., N. A., Jan. 10, 1918. Discharged Dec. 11, 1918. San. Insp. at Camp Greenleaf, Ft. Oglethorpe, Camp J. E. Johnston.

VOLK, WENDELL DOUGLAS
Entered service Sept. 2, 1917; 1st Lt., Engrs., N. A., July 26, 1917; Capt., Engrs., N. A., July 30, 1918. Overseas service Nov. 11, 1917–Sept. 11, 1918. Discharged Aug. 18, 1919. With 20th Engrs., A. E. F., as Acting Co. Comdr., Police Officer, and Supply Officer; Garrison School, 5th Engrs.; Adj., 20th Engrs., Camp Forrest.

von DEESTEN, ARTHUR PETER
Entered service Jan. 12, 1916, as 2d Lt., C. of E., U. S. A.; 1st Lt., June, 1916; Capt., July, 1917; Maj., Mar. 19, 1918. Co. Comdr., 5th Engrs.; in chg. ponton school, Camp Humphreys; Bn. Comdr., 217th Engrs.; post-graduate student, engr. school.

WADE, GEORGE WILLIS
Entered service Aug. 27, 1917; 1st Lt., Engrs., N. A., Sept. 1, 1917; Capt., Engrs., U. S. A., May 1, 1918. Overseas service Oct. 18, 1917–Apr. 28, 1919. Discharged May 16, 1919. Co. Comdr., Bn. Adj., and Personnel Officer, 117th Engrs. Five stars.

WADSWORTH, GEORGE REED
Entered service June 13, 1917, as Capt., Sig. R. C.; Maj., A. S., N. A., Oct. 3, 1917. Discharged Dec. 30, 1918. Chf. Engr., Naval Aircraft Factory, Philadelphia Navy Yard.

WADSWORTH, HENRY HAYES
Entered service Mar. 1, 1917, as Maj., E. O. R. C. Released from active service Aug. 14, 1917. River and harbor work.

WAGNER, ALLAN JOHN
Entered service Oct. 21, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 31, 1918. E. O. T. S., Camp Humphreys.

WAGNER, JOHN, JR.
Entered service May 11, 1917; 1st Lt., Cavalry, N. A., Aug. 15, 1917; Capt., P. M. G. D., May 14, 1919. Overseas service Sept. 11, 1917–June 28, 1919. Discharged July 18, 1919. Instr., 1st Corps School, A. E. F.; A. P. M., Autun, France.

WAITE, CLEMENT F.
Entered service Sept. 2, 1917, as 2d Lt., E. O. R. C.; 1st Lt., Engrs., N. A., June 19, 1918; Capt., Engrs., U. S. A., Oct. 28, 1918. Discharged Mar. 6, 1919. With 316th Engrs., Camp Lewis; with 318th Engrs., Vancouver Barracks; with 3d Engr. Training Regt., Camp Humphreys; with 2d Engr. Training Regt. as Personnel Adj., Camp Humphreys; Personnel Adj. and Co. Comdr., 220th Engrs.

WAITE, HENRY MATSON
Entered service Jan. 29, 1918, as Lt. Col., Engrs., N. A.; Col., T. C., U. S. A., Oct. 4, 1918. Overseas service Feb. 7, 1918–Mar. 1919. Discharged Mar., 1919. Engr. Constr., T. C., A. E. F.; Deputy Director, Transport, 2d Army and 3d Army; Civil Affairs, Advance G. H. Q., Trier, Germany. Distinguished Service Medal; Officier, Legion d'Honneur. Two stars.

WALDRON, ALBERT EDWIN
Entered service June 19, 1895; through all grades in C. of E., U. S. A., to Col., Oct. 1, 1917. Overseas service Jan. 29, 1918–Feb. 25, 1919. Reverted to permanent rank of Lt. Col., C. of E., U. S. A., July 14, 1919. C. O., 35th Engrs., ry. shop unit. Officier, Legion d'Honneur.

WALKER, EDWARD GEORGE
Entered service Nov. 14, 1916, as Lt., Royal Naval Volunteer Reserve, Great Britain; Capt., Royal Air Force, Apr. 1, 1918. Discharged Aug. 31, 1919. In Technical Branch, Directorate of Balloons, in chg. development kite balloon design.

WALKER, ELTON DAVID
Entered service May 8, 1917; Capt., E. O. R. C., Mar. 21, 1917. Overseas service July 9, 1917–Jan. 25, 1919. Discharged Jan. 30, 1919. Co. Comdr., 15th Engrs., in misc. constr. work in Advance Sec., A. E. F.; Water Supply Service, Div. of Constr. and Forestry, S. O. S., A. E. F.; with Sec. Engr., Base Sec. No. 4, in chg. water supply and sanitation. Citation from Gen. Pershing for meritorious service.

WALKER, HARRY BRUCE
Entered service Sept. 2, 1917; Capt., E. O. R. C., July 3, 1917. Overseas service May 27, 1918–June 6, 1919. Discharged June 28, 1919. With 303d Engrs.; Asst. Div. Engr., 78th Div.; St. Mihiel and Meuse-Argonne offensives. Two stars.

WALKER, JOHN PALMER
Entered service Oct. 10, 1918, as Capt., Engrs., U. S. A. Discharged Jan. 13, 1919. With 420th Engrs.; with 3d Engr. Training Regt., Camp Humphreys.

WALKER, MERIWETHER LEWIS
Entered service June 15, 1889; through all grades in C. of E., U. S. A., to Lt. Col. at declaration of war; Col., Engrs., N. A., Aug. 5, 1917; Brig. Gen., U. S. A., June 26, 1918. Overseas service Nov. 26, 1917–Aug. 23, 1919. Returned to permanent rank of Lt. Col. C. of E., U. S. A., Aug. 31, 1919. C. O., 116th Engrs.; Engr. Supply Officer, A. E. F.; Director M. T. C., A. E. F. Distinguished Service Medal; Officier, Legion d'Honneur.

WALKER, WILLIAM KEMP
Entered service Dec. 28, 1917, as Capt., Engrs., N. A. Discharged Feb. 26, 1919. With Director Gen., Military Rys., in chg. Western Dist., Chicago, Ill.

WALLACE, DAVID ALEXANDER
Entered service Sept. 5, 1917; 1st Lt., Engrs., N. A., July 15, 1917; Capt., Engrs., N. A., Dec. 11, 1917. Overseas service Jan. 26, 1918–Oct. 22, 1918. Discharged Feb. 8, 1919. With 311th Engrs.; Dist. Engr., Port Dept., Office, Chf. Engr., Transportation, A. E. F. One star.

WALLACE, HAROLD ULMER
Entered service Mar. 6, 1918, as Maj., Q. M. C., N. A. Discharged Apr. 1, 1919. Superv. Constr. Officer, Aberdeen Proving Grounds, Edgewood Gas Plant, Mays Landing Shell-Loading Plant, Marlin Rockwell Shell-Loading Plant, Hastings Gas Plant, Lakehurst Proving Grounds and Training Camp, Western Cartridge Plant and LaClede Shell-Loading Plant.

WALLER, ALEXANDER CHARLES
Entered service as 1st Lt., Ord. R. C., Nov. 27, 1917; 1st Lt., Engrs., N. A., July 29, 1918. Overseas service Sept. 17, 1918–July 2, 1919. Discharged July 10, 1919. With 22d Engrs. One star.

WALLER, PERCY
Entered service Dec. 28, 1917; Capt., Engr. R. C., Sept. 26, 1917. Overseas service Sept. 5, 1918–June 23, 1919. Discharged July 14, 1919. With 22d Engrs.; Div. Supt. of Operation, Light Rys., Meuse-Argonne offensive. One star.

WALTER, FRANK EDGAR
Entered service June 20, 1918, as 1st Lt., Engrs., N. A.; Capt., Engrs., U. S. A., May 11, 1919. Overseas service June 30, 1918–July 5, 1919. Discharged July 9, 1919. With 55th Engrs. One star.

WALTON, HARRISON BILLINGSLEY
Capt., Engrs., U. S. A., A. E. F.*

WAND, ANTHONY WILLIAM
Entered service May 12, 1918, as Pvt., Arty., N. A.; Sgt., Engrs., U. S. A., July 1, 1919; 2d Lt., Engrs., U. S. A., Oct., 1919. Discharged Dec. 7, 1919. With 1st Replacement Regt. Camp Humphreys.

WANZER, JAMES OLIN
Entered service May 13, 1917; 1st Lt., Engrs., N. A., June 28, 1918; Capt., T. C., U. S. A., May 13, 1919. Overseas service July 14, 1918–Aug. 23, 1919. Discharged Sept. 13, 1919. With 47th Engrs.; Camp Constr. Officer, Camp No. 7, Montierchaume, France; Chf., R. T. O., 3d Army; Adj., 24th Grand Div., T. C. One star.

WARD, GEORGE SPARKMAN
Entered service July 25, 1917, as Pvt., Engrs., S. C. N. G.; Sgt., Engrs., N. A., Aug. 1, 1917; M. E., Sr. Grade, Engrs., N. A., Sept. 18, 1917; 2d Lt. Engrs., N. A., Oct. 25, 1917 Overseas service Oct. 17, 1917–Apr. 28, 1919. Discharged May 3, 1919. With 117th Engrs., 42d Div.

WARD, JASPER DUDLEY
Entered service July 3, 1917, as Pvt., Inf., U. S. A.; Cpl., Inf., Aug. 27, 1917; Sgt., Inf., Apr. 18, 1918; 2d Lt., Inf., June 10, 1918; 1st Lt. Inf. Sept. 9, 1918. Discharged Dec. 7, 1918. With 55th Inf. and 162d Depot Brig.

WARD, LYMAN WISE
Capt., C. A. C., U. S. A.*

WARDWELL, RALPH WATTS
Entered service Oct. 21, 1918, as Capt., Engrs., U. S. A. Discharged Oct. 28, 1919. In training, Camp Humphreys; with C. A. C., Ft. Monroe.

WARE, HOWARD THOMAS
1st Lt., Q. M. C., U. S. A.*

WARE, JOHN
Entered service Aug. 5, 1917, as 1st Lt., Engrs., N. A.; Capt., Engrs, N. A., July 29, 1918. Overseas service Sept. 26, 1917–Apr. 4, 1919. Discharged May 17, 1919. With 101st Engrs., in chg. procuring and distributing eng. equipment, 26th Div. Four stars.

WARE, NORTON
Entered service Sept. 25, 1917; Capt., E. O. R. C., Aug. 15, 1917; Maj., Engrs., U. S. A., July 30, 1918. Overseas service Jan. 21, 1918–Sept. 19, 1919. Discharged Oct. 28, 1919. With 1st Engrs. One star.

WARFIELD, RALPH MERVINE
Entered service July 30, 1907, as Asst. C. E., U. S. N.; Lt. Comdr., C. E. C., U. S. N., July 1, 1917. Public Works Officer, Naval Air Station, Pensacola, Fla.

WARING, CHARLES THOMAS
Entered service June 19, 1917, as Capt., Sig. R. C.; Maj., A. S., U. S. A., Aug. 15, 1917. Overseas service Sept. 3, 1918–Dec. 4, 1918. Discharged June 16, 1919. Officer in chg. of constr., Wilbur Wright Aviation Field; temporary head, Constr. Div. Sig. C., Washington, D. C.; in chg. six aviation fields in Texas; in chg. constr. Langley Field; C. O. Aircraft Acceptance Park No. 2; C. O., Supply Depot, Wilbur Wright Flying Field.

WARNER, ELWIN STREETER
Entered service May 8, 1917; 1st Lt., E. O. R. C., Apr. 2, 1917; Capt., Engrs., N. A., Aug. 15, 1917; Maj., Engrs., U. S. A., Mar. 26, 1919. Overseas service July 14, 1918–June 13, 1919. Discharged July 5, 1919. With 301st Engrs., div. engrs. of 76th Div. and Corps Engrs. with 4th Army Corps; St. Mihiel offensive; with 3d Army.

WARNOCK, WILLIAM HAROLD
Entered service Apr. 3, 1918, as Capt., Q. M. C., Constr. Div., N. A. Discharged Sept. 6, 1919. Field Officer to Const. Q. M., Camp Stuart; Const. Q. M., and Disbursing Officer, Skiff's Creek Water Development.

WARREN, HORACE PRETTYMAN
Entered service Apr. 16, 1918; Maj., Engrs., N. A., Apr. 20, 1918; Lt. Col., Engrs., U. S. A., Feb. 13, 1919. Overseas service June 30, 1918–Sept. 3, 1919. Discharged Sept. 24, 1919. Bn. Comdr. and C. O., 55th Engrs.; Sec. Engr., Intermediate Sec.; Officer in chg. constr. Pershing Stadium. Citation from Gen. Pershing for meritorious service.

WARREN, JAMES GOOLD
Entered service July 1, 1877; through all grades in C. of E., U. S. A., to Col. at declaration of war. On duty with engr. troops, river and harbor works, fortifications, etc.

WARREN, MINTON MACHADO
Entered service Sept. 17, 1917; 1st Lt., Engrs., N. A., Sept. 22, 1917; Capt., Engrs., U. S. A., Aug. 18, 1918. Overseas service Oct. 9, 1917–Apr. 4, 1919. Discharged Apr. 28, 1919. Co. Comdr., 101st Engrs.; Topographical Officer, 26th Div.; organized 1st American Div., Topographical Sec.; road repair work. Five stars.

WARREN, PHILIP RIDSDALE
Entered service Oct. 1, 1917, as 1st Lt., Royal Engrs., British Army; Maj., Royal Engrs., Dec. 15, 1917; Lt. Col., Royal Engrs., Mar. 1, 1919. Overseas service Dec. 15, 1917–Oct. 7, 1919. Discharged Oct. 7, 1919. Port Constr. Engr. in France; engr. in chg. reconstr. Belgian ports; Asst. Director Gen. of Transportation, France. Order of the British Empire; Officer, Order of the Crown, Belgium.

WARREN, WILLIS DOW PECK
Entered service Sept. 11, 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec. 17, 1918. E. O. T. S., Camp Humphreys; with 139th Engrs., Camp Shelby.

WASHINGTON, WALTER OWEN
Entered service May 8, 1917; Capt., E. O. R. C., Jan. 23, 1917. Discharged Dec. 11, 1917. Camp Funston and Ft. Sam Houston.

WATERMAN, EARLE LYTTON
Entered service Aug. 10, 1918; 1st Lt., San. C., N. A., July 27, 1918; Capt., San. C., U. S. A., June 6, 1919. Discharged Aug. 25, 1919. Camp San. Engr., Camps Greene and Meade.

WATKINS, GUY ANDERSON
Entered service Dec., 1917, as Capt., Q. M. C., N. A. Discharged Feb. 7, 1919. Asst. to Const. Q. M., Camps Lee and Bragg; Asst. Const. Q. M., Camp Las Casas, San Juan, Porto Rico.

WATSON, DAVID LOYALL FARRAGUT
Entered service Oct. 25, 1918, as 1st Lt., Engrs., U. S. A. Discharged Apr. 7, 1919. E. O. T. S., Camp Humphreys; with 2d Engr. Training Regt. and 5th Engrs., Camp Humphreys.

WATSON, DAVID MOWAT
Lt., Royal Arty. B. E. F.*

WATSON, GEORGE LINTON
Entered service Sept. 25, 1917; Capt., Engrs., N. A., Oct. 25, 1917; Maj., Engrs., N. A., June 16, 1918; Lt. Col., Engrs., U. S. A. Oct. 9, 1918. Overseas service Nov. 25, 1917–Mar. 25, 1919. Discharged Oct. 9, 1919. With 30th Engrs. and British Army at Lens Sector, Portuguese Corps in Estaire Sector, Belgian Army in Ypres Sector and French Army in Champagne Sector; Asst. G-2, 1st Army Corps and 3d Army; A. C. of S. Advance G. H. Q., Treves, Germany; duty with War Plans Div., Gen. Staff; G. S. C.; served in eight major engagements. Wounded three times: High explosive at Lens; shrapnel at Merville; gassed in St. Mihiel operations. Cited in orders of 26th Div., 6th Div., 8th French Army, 32d French Army Corps, 1st British Army; Officier, Legion d'Honneur; Military Cross, Great Britain; Croix de Guerre with Star and Palm; Order of the Crown, Belgium. Six stars.

WATSON, WINSLOW BARNES
Entered service July, 1916, as 2d Lt., Inf., N. Y. N. G.; 1st Lt., Inf., N. Y. N. G., July, 1917; Capt., Inf., U. S. A., Oct. 26, 1918. Overseas service May 8, 1918–Mar. 6, 1919. Discharged Apr. 2, 1919. Co. Comdr., 106th Inf. Divisional citation. Two stars. Wounded.

WATT, DAVID ALEXANDER
Entered service Sept. 6, 1917; Maj., E. O. R. C., Feb. 23, 1917. Discharged May 29, 1919. Attached to Office, Chf. of Engrs., Washington, D. C.; in chg. field constr., Florence, Ala.

WAUGH, WILLIAM HAMMOND
Entered service June 13, 1917, as Capt., E. O. R. C.; Maj., Engrs., N. A., June 11, 1918. Pres. and Engr. Officer, Alaska Road Commission.

WAY, WILLIAM FLOYD
Entered service June 12, 1917; Sgt., 18th Engrs., June 20, 1917; 2d Lt., Engr. R. C., May 8, 1918; 1st Lt., Engrs., U. S. A., Sept., 1918. Overseas service Aug. 9, 1917–July 2, 1919. Discharged July 29, 1919. With 18th Engrs. in dock and ry. constr., Base Sec. No. 2, A. E. F.; with Div. of Constr. and Forestry in work on Engr. Valuation Bd., Tours, France.

WEAVER, CHARLES JOSEPH
Entered service Sept. 25, 1917, as 1st Lt., Engrs., N. A. Overseas service Feb. 15, 1918–July 18, 1919. Discharged Oct. 6, 1919. With 301st Tank Bn.; Instr. U. S. Tank Center,

Wareham, Dorset, England; with 21st Engrs., light ry. constr. and maintenance in Meuse-Argonne offensive. Three stars.

WEAVER, EARLL CHASE
Entered service Apr. 10, 1917; Lt., Jr. Grade, C. E. C., U. S. N. R. F.; Lt., C. E. C., U. S. N. R. F., June 18, 1918. Released from active service Apr. 17, 1920. Asst. to Public Works Officer, Puget Sound Navy Yard.

WEAVER, FRANK LLOYD
Entered service May 14, 1917; 1st Lt., E. O. R. C., June 19, 1917; Capt., Engr. R. C., May 12, 1918. Overseas service May 26, 1918–June 3, 1919. Discharged June 30, 1919. With 305th Engrs., Artois Sector, with British Army; St. Mihiel and Meuse-Argonne offensives. Three stars.

WEBB, CLAUDE ALLEN
Entered service Aug. 27, 1917; 2d Lt., F. A., N. A., Nov. 27, 1917; 1st Lt., F. A., U. S. A., Nov. 8, 1918. Overseas service Dec. 24, 1917–May 14, 1919. Discharged May 17, 1919. Saumur, France, Arty. School; with French as observer; with 121st F. A., at Rouge Mont, 2d Battle of Marne, Soissons and Meuse-Argonne.

WEBB, DeWITT CLINTON
Entered service June, 1903; through all grades in C. E. C., U. S. N., to Lt. Comdr. at declaration of war and Comdr., Feb. 1, 1918. Public Works Officer, Boston and Philadelphia Navy Yards.

WEBB, GEORGE HERBERT
Entered service June 17, 1917; Maj., E. O. R. C., May 26, 1917; Lt. Col., E. O. R. C., July 6, 1917; Col., Engrs., U. S. A., Sept. 27, 1918. Overseas service Aug. 1, 1917–Apr. 6, 1919. Discharged Apr. 14, 1919. Bn. Comdr. and C. O., 16th Engrs.; Sec. Engr., Intermediate Sec. West, A. E. F. Distinguished Service Medal; Officier, Order de l'Etoile Noire.

WEBB, ISHAM GANO
Capt., Engrs., U. S. A.*

WEBB, WALTER LORING
Entered service Oct. 17, 1917; Maj., E. O. R. C., Feb. 12, 1917. Overseas service Oct. 29, 1917–Jan. 24, 1920. On staff, Director Gen. of Transportation, A. E. F.; Chf. Engr. of Renting, Requisitions and Claims Service, A. E. F., in chg. valuation of claims.

WEBER, CHARLES MARIA
Entered service June, 1918; Chf., Q. M. Aviation, U. S. N. R. F., Aug. 1918; Ensign, U. S. N. R. F., June 23, 1919. Released from active service June 24, 1919. Ground School at Boston; flying at Miami and Pensacola, Fla.

WEBSTER, MAURICE ANDERSON
Entered service Aug. 6, 1917, as 1st Lt., Ord. R. C.; Capt., Ord. C., N. A., Jan. 15, 1918. Discharged Jan. 13, 1919. At Aberdeen and Sandy Hook Proving Grounds; Ft. Hancock, ord. testing.

WEEKS, WILLIAM CHARLES
Entered service Mar. 7, 1918; Maj., Engrs., N. A., Mar. 14, 1918; Lt. Col., Engrs., U. S. A., Nov. 2, 1918. Overseas service June 15, 1918–June 9, 1919. With 32d Engrs., gen. constr. work in Base Sec. No. 2, A. E. F.; Asst. to Dept. Engr., Southern Dept.

WEIDMAN, WILLIAM ROE
Entered service May 20, 1918; Capt., Engr. R. C., May 7, 1918. Overseas service Oct. 27, 1918–July 11, 1919. Discharged Aug. 4, 1919. With 5th Engr. Training Regt.; with 547th Engrs. and with 10th Engr. Service Co.

WELBORN, MARVIN CURTIS
Entered service Oct. 28, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 11, 1918. E. O. T. S., Camp Humphreys.

WELDEN, ERNEST
Entered service Apr. 7, 1918, as Pvt., Engrs., N. A.; Sgt., Engrs. Overseas service May 21, 1918–July 4, 1919. Discharged July 10, 1919. With 43d Bn., attached to 20th Engrs.

WELLES, THEODORE LADD, JR.
Entered service Sept. 25, 1917, as 2d Lt., Engr. R. C.; 1st Lt., Engrs., U. S. A., Oct. 30, 1918. Overseas service May 8, 1918–June 11, 1919. Resigned Aug. 2, 1919. With 318th Engrs.; with 314th Engrs. in St. Mihiel and Meuse-Argonne offensives. Two stars.

WELLS, EMERY
Entered service May 8, 1917; 1st Lt., E. O. R. C., June 11, 1917. Overseas service Dec. 11, 1917–Feb. 12, 1918. Discharged May 30, 1919. With 314th Engrs. at Camp Funston; Asst. Depot Engr. Officer, Advance Engr. Depot, Is-sur-Tille, France. One star.

WELLS, JAMES BERTRAND
Entered service Sept. 18, 1917, as Pvt., Inf., N. A. Discharged Mar., 1918. With 363d Inf. at Camp Lewis.

WELSH, RUSSELL DUTTON
Entered service May 28, 1918, as Pvt., Engrs., N. A.; 2d Lt., Engrs., U. S. A., Mar. 26, 1919. Overseas service Aug. 8, 1918–July 5, 1919. Discharged July 24, 1919. With 115th Engrs., 40th Div., and with 3d Army, Coblenz, Germany.

WENIGE, ARTHUR EMIL
Maj., Engrs., U. S. A.*

WENZELL, ANDREW PERRY
Entered service Apr. 8, 1917; Capt., E. O. R. C., May 13, 1917; Maj., Engrs., U. S. A., Aug. 9, 1918, and Lt. Col., Engrs., U. S. A., Feb. 13, 1919. Overseas service July 1, 1917–May 28, 1919. Discharged June 1, 1919. With 16th Engrs., Lys defensive and Meuse-Argonne offensive. Two stars.

WEST, EDWARD HAZZARD
Entered service Sept. 2, 1917, as Capt., E. O. R. C.; Maj., Engrs., U. S. A., Oct. 1, 1918. Overseas service July 31, 1918–Feb. 25, 1919. Discharged Mar. 26, 1919. Regtl. Adj. and Bn. Comdr., 5th Engrs. One star.

WEST, WADE CLARENCE
Entered service June 22, 1918; Capt., E. O. R. C., Sept. 21, 1917. Discharged Jan. 10, 1919. Constr. Officer, Prison Officer, Co. Comdr., 5th Engr. Training Regt., Camp Humphreys.

WESTOVER, HENRY CHRISTOPHER
Entered service Sept. 13, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 5, 1918. Unattached, special service.

WHEAT, GEORGE NEVILLE
Entered service Sept. 27, 1917; Capt., Engr. R. C., Aug. 8, 1917. Overseas service Jan. 27, 1918–Jan. 13, 1919. Discharged Jan. 18, 1919. With 113th Engrs.; Transportation Dept., Bourges, France; Div. of Constr. and Forestry, S. O. S., A. E. F.; with Sec. Engr., Base Sec. No. 7.

WHEELER, EDGAR TRUE
Entered service Oct. 7, 1918; Capt., Engrs., U. S. A., Nov., 1918. Discharged Jan. 8, 1919. With 125th Engrs.

WHEELER, FRANK IGNATIUS, JR.
Entered service Aug. 1, 1917, as Pvt., F. A., Md. N. G.; 2d Lt., Aviation Sec., Sig. R. C., Nov. 8, 1917; 1st Lt., A. S. A. P., July 16, 1918. Discharged May 1, 1919. With Equipment Div., Aviation Sec., Sig. C., in chg. aircraft armament manufacture; Chf. Insp., Tools and Machinery Sec., Inspection Dept., Production Div., Bureau Aircraft Production.

WHEELER, ROBERT CLARK
Entered service Aug. 13, 1918, as Capt., Q. M. C., Constr. Div., U. S. A.; Maj., Q. M. C., Constr. Div., U. S. A., June 16, 1919. Discharged Mar. 2, 1920. Asst. Port Utilities Officer, Newport News, Va.; in chg. Water Supply Sec., Constr. Div.

WHEELOCK, DE FOREST AUGUSTUS
Col., U. S. A.*

WHITEAKER, ROBERT ORLANDO
Entered service Mar. 31, 1917; Capt., Cavalry, Texas N. G., Mar. 17, 1917; Capt., F. A., N. A., Oct. 15, 1917. Overseas service July 4, 1918–Dec. 24, 1918. Battery Adjt. and Comdr. 132d F. A.; Operations Officer 132d F. A.

WHITMAN, EZRA BAILEY
Maj., Q. M. C., U. S. A.*

WHITMAN, RALPH
Entered service Aug. 11, 1907, as Ensign, C. E. C., U. S. N. Through all grades to Lt. Comdr., C. E. C., U. S. N., July 1, 1917. Overseas service Apr. 19, 1917–Apr. 23, 1920, in Santo Domingo. Aide on staff of Military Governor, Santo Domingo. Special letter of commendation from Secretary of Navy. Silver star.

WHITNEY, CHARLES SMITH
Sgt., Engrs., U. S. A., A. E. F.*

WHITNEY, JOHN THAD
Entered service Aug. 27, 1917; 2d Lt., F. A. R. C., Nov. 27, 1917. Overseas service July 10, 1918–July 6, 1919. Discharged July 11, 1919. With 341st F. A.; F. A. Replacement Depot, Camp Jackson; 2d Corps Arty. Park, A. E. F.; Motor Transport Office, Brest, France. Five stars.

WHITNEY, RALPH EDWARD
Capt., San. C., U. S. A.*

WHITSIT, LYLE ANTRIM
Entered service Oct. 24, 1918, as Capt., Engrs., U. S. A. Discharged Aug. 27, 1920. Power Sec., War Industries Bd.

WHITWELL, EDWARD
Capt., Royal Air Force, British Army.*

WICKERSHAM, JOHN HOUGH
Entered service May 9, 1917; Capt., E. O. R. C., June 5, 1917; Maj., Engrs., N. A., July 14, 1918; Lt. Col., Engrs., U. S. A., Nov. 9, 1918. Overseas service July, 1917–Jan., 1919. Discharged Feb. 8, 1919. Requisitioning supplies for Line of Communications, A. E. F., and for Chf. Engr., A. E. F.; Deputy Engr. Supply Officer, A. E. F.; representative, Chf. of Engrs. on 4th Sec., Gen. Staff, G. H. Q., A. E. F. Two citations from C. in C., A. E. F.; letter of commendation from C. of E., U. S. A. Four stars.

WIDDICOMBE, ROBERT ALEXANDER
Maj., Engrs., U. S. A.*

WIDDICOMBE, STACEY HARRISON
Entered service Jan. 29, 1918, as Pvt., 1st Class, A. S., N. A. Discharged Dec. 16, 1918. Ground Schools, Princeton and Camp Dick; School of Fire, Ft. Sill; School of Artillery Observation, Ft. Sill.

WIGGIN, THOMAS HOLLIS
Entered service May 16, 1917, as Capt., E. O. R. C.; Maj., Engrs, N. A., July 10, 1918; Lt., Col., Engrs., U. S. A., Apr. 7, 1919. Overseas service July 28, 1917–June 7, 1919. Discharged July 30, 1919. On staff of Chf. Engr., Line of Communication, A. E. F.; and Chf. Engr., A. E. F., in Div. of Constr. and Forestry; in chg. Water Supply Sec. Citation from Gen. Pershing for meritorious service; Chevalier, Merite Agricole.

WIGGINS, RALPH RAYMOND
Entered service Aug. 27, 1917; 1st Lt., A. S., N. A., Nov. 8, 1917. Discharged July 14, 1919. Asst. to Officer in Chg. Constr., and Officer in Chg. Constr., Hazlehurst and Mitchel Fields.

WILCOX, ERNEST HARDWICK
Entered service Sept. 7, 1917, as Capt., E. O. R. C.; Overseas service Sept. 18, 1918–June 20, 1919. Discharged July 28, 1919. C. O., 543d Engrs.; with 603d Engrs. One star.

WILD, HERBERT JOSEPH
Entered service May 5, 1917; Capt., E. O. R. C., Feb. 12, 1917; Maj., Engrs., N. A., July 20, 1918. Office, Chf. of Engrs.; Instr., 2d training camp; attached 305th Engrs.; with 3d Engrs. in Canal Zone; with 220th Engrs.; Prof. of Military Science, Missouri School of Mines.

WILGUS, HERBERT SEDGWICK
Entered service Sept. 10, 1918, as Maj., Engrs., U. S. A. Discharged Dec. 18, 1918. Bn. Comdr., 138th Engrs.

WILGUS, WILLIAM JOHN
Entered service May 10, 1917; Maj., E. O. R. C., Feb. 16, 1917; Col., R. T. C., N. A., Oct. 26, 1917. Overseas service May 14, 1917–Jan. 1, 1919. Discharged Jan. 2, 1919. Member Military Ry. Comm. to England and France; Director of Rys., A. E. F.; Deputy Director Gen. of Transportation, A. E. F. Distinguished Service Medal; Officier, Legion d'Honneur.

WILLARD, GEORGE THOMPSON
Entered service Dec. 13, 1917, as Pvt., Engrs., N. A.; Sgt., Engrs., U. S. A., Aug., 1918. Overseas service May 7, 1918–July 6, 1919. Discharged July 11, 1919. With 318th Engrs.,; Instr., 3d Corps School, A. E. F.

WILLARD, NORMAN
Entered service Sept. 25, 1917; Capt., Engrs., N. A., Aug. 25, 1917; Overseas service Mar. 22, 1918–Aug. 1, 1919. Discharged Aug. 22, 1919. Co. Comdr., 510th Engrs.; Personnel Adj. to Sec. Engr., Base Sec. No. 1, A. E. F.; Executive Officer, A. T. S., Antwerp, Belgium.

WILLCOMB, GEORGE EDWARD
Entered service Aug. 19, 1918, as Capt., Q. M. C., Constr. Div., U. S. A. Discharged Mar. 25, 1919. Asst. to Utilities Officer, Camp Wadsworth.

WILLCOX, HENRY
Entered service Apr. 22, 1918, as 1st Lt., Q. M. C., Constr. Div., N. A.; Capt., Q. M. C., Constr. Div., U. S. A., Sept. 5, 1918. Discharged Jan. 14, 1919. Asst. to Const. Q. M., Augusta Arsenal Depot, Mechanical Repair Unit No. 305, and Ft. McPherson.

WILLIAMS, ALAN FRANK
Entered service June 13, 1917, as Pvt., Engr. R. C.; M. E., Sr. Grade, Engrs., N. A., July 15, 1917; 2d Lt., Engrs., N. A., Mar. 17, 1918. Overseas service Aug. 9, 1917–Apr. 28, 1919. Discharged June 6, 1919. With 18th Engrs. on dock and railroad constr. in Base Sec. No. 2, A. E. F.; graduate, Army School of the Line and Gen. Staff College, Langres, France.

WILLIAMS, GARDNER STEWART
Entered service July 22, 1917; Maj., E. O. R. C., Jan. 23, 1917. Relieved from active service Dec. 17, 1917; resigned Apr. 19, 1918. On Const. Q. M. staff, Camp Beauregard; Bd. to appraise damages, Camp Beauregard; Constr. Div. H. Q., Washington, D. C.

WILLIAMS, GEORGE DAVID
Entered service May 1, 1918; Capt., Engrs., N. A., Feb. 9, 1918. Overseas service Oct. 26, 1918–July 7, 1919. Discharged July 14, 1919. With 548th Engrs. on forestry and highway work in Cote d'Or, France.

WILLIAMS, SAMUEL WALTER
Entered service Sept. 2, 1917, as Capt., Engrs., N. A.; Maj., Engrs., N. A., Dec. 30, 1917. Overseas service June 25, 1918–Jan. 25, 1919. Discharged Jan. 30, 1919. With 315th Engrs.; Asst. to Div. Engrs., 90th Div.; C. O., 531st Engrs.; with Sec. Engr., Intermediate Sec. West, A. E. F.

WILLIAMSON, HARRY
Entered service Jan. 3, 1915, as Lt., Royal Engrs., British Army; Capt., Royal Engrs., Dec. 15, 1917; Maj., Royal Engrs., June 1, 1918. Overseas service Jan. 3, 1916–Aug. 30, 1919. Released from active service Sept. 1, 1919. With 120th Co. and 272d Ry. Co. in Ypres Sector, and with Egyptian Expeditionary Force in ry. constr. Mentioned in despatches. Wounded once.

WILLIAMSON, LEE HOOMES
Entered service June 17, 1918, as Lt., Engrs., N. A. Overseas service June 30, 1918–Apr. 1, 1919. Discharged Apr. 5, 1919. Co. Comdr., 55th Engrs.; Asst. Supply Officer, Intermediate Sec. West, A. E. F.; Co. Comdr., 122d Engrs.; Engr. Officer in chg. constr. Belgian Camp, Le Mans, France.

WILLIAMSON, SYDNEY BACON
Entered service June 18, 1918, as Lt. Col., Engrs., N. A.; Col., Engrs., N. A., Sept. 19, 1918. Overseas service July 8, 1918–Mar. 20, 1919. Discharged Mar. 21, 1919. Sec. Engr., Intermediate Sec. West, and Paris Dist., A. E. F.

WILLIAR, HARRY DUGAN, JR.
Maj., Engrs., U. S. A., A. E. F.*

WILLIS, WALTER JOHN
Entered service Apr. 6, 1917, as Lt., N. N. V. Released from active service Dec. 31, 1919. Engr. Aide to Industrial Mgr., Navy Yard, New York; Outside Ship Supt., Machinery Div., Navy Yard.

WILSON, EDBERT CARSON
Entered service June 12, 1918; Capt., Engr. R. C., June 7, 1918. Overseas service Sept. 29, 1918–May 2, 1919. Discharged May 14, 1919. Co. Comdr., 2d Engrs. Training Regt. Camp Humphreys; Co. Comdr. 211th Engrs., Camp Forrest; Water Supply Officer, 2d Army Corps; special duty with Water Supply Officer, 9th British Corps; Co. Comdr., 114th Engrs.

WILSON, EVERITT WYCHE
Entered service Aug. 25, 1917, as Capt., Engrs., N. A. Overseas service Mar. 22, 1918–Sept. 9, 1919. Discharged Sept. 25, 1919. In chg. of water supply operation, Base Sec. No. 1, A. E. F.; attached to 17th Engrs.

WILSON, HARRY PERCIVAL
Capt., Engrs., U. S. A., A. E. F.*

WILSON, LOUIS ARTHUR
Entered service Oct. 4, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 3, 1918. Co. Comdr., 417th Engrs, in training.

WILSON, NORMAN KENNETH
Entered service Dec. 12, 1917; 2d Lt., A. S., N. A., Nov. 24, 1917. Overseas service Jan. 24, 1918–Dec. 11, 1918. Discharged Dec. 22, 1918. With 473d Aero Squadron; Water Supply constr. Romsey, England, A. S. Camp; in chg. constr., water supply, sewage disposal, railroads, and airdrome at Rustington, England.

WILSON, ROBERT BROWN MURPHY
Entered service May 13, 1917; Capt., Engrs., N. A., Aug. 15, 1917; Maj., Engrs., U. S. A., Aug. 26, 1918. Overseas service Sept. 9, 1918–June 27, 1919. Discharged July 24, 1919. With 311th Engrs., 86th Div.

WILSON, WILLIAM RENFREW
Entered service Apr. 24, 1915; 2d Lt., Royal Engrs., British Army, June, 1915; Capt., Royal Engrs., Jan. 16, 1916; Maj., Royal Engrs., Nov., 1916; Lt. Col., Royal Engrs., Aug., 1918. Overseas service June, 1915–Jan., 1920. Discharged Jan. 13, 1920. With 23d Field Co.; with 40th Div. as C. R. E.; in Siberia with British Ry. Mission. Military Cross, Great Britain; Croix de Guerre; mentioned in despatches.

WINCHESTER, THOMAS HARRISON
Entered service Aug. 17, 1918; 1st Lt., Engrs., E. O. T. S., Aug. 5, 1918. Discharged Dec. 20, 1918. With 9th Engr. Training Regt.

WING, CHARLES BENJAMIN
Entered service Sept. 2, 1917; Maj., E. O. R. C., June 14, 1917; Lt. Col., Engrs., U. S. A., Oct. 6, 1918. Overseas service Mar. 30, 1918–June 12, 1919. Discharged June 14, 1919. With 23d Engrs.; on constr. Nevers, France, railroad cut-off; directed constr. Chateauroux Storage Depot, A. E. F.; constr. and maintenance of roads, 1st Army; maintenance and reconstruction roads in St. Mihiel and Meuse-Argonne areas. One star.

WINN, WALTER EDWARD
Entered service May 14, 1917; Maj., E. O. R. C., June 19, 1917; Lt. Col., Engrs., N. A., Nov. 24, 1917. Overseas service Aug. 22, 1918–Nov. 1, 1918. Discharged Feb. 13, 1919. 2d in command and C. O., 114th Engrs.; Asst. Chf. of Staff, 39th Div. in chg. of instruction in field fortifications, hand grenades, and map reading.

WINN, WALTER SCOTT
Maj., Engrs., U. S. A.*

WINTON, WALTER FERRELL
Entered service Oct. 7, 1911, as 2d Lt., F. A., U. S. A.; Capt., F. A., U. S. A., May 15, 1917; Maj., F. A., U. S. A., July 3, 1918; Lt. Col., F. A., U. S. A., Oct. 26, 1918. Overseas service May 27, 1918–July 16, 1919. Returned to permanent rank Mar. 15, 1920. Served with 19th F. A., 5th Arty. Brig.; St. Mihiel and Meuse-Argonne offensives. Two stars.

WODRICH, OSCAR FREDERICK
Entered service July 24, 1918, as Capt., Q. M. C., Constr., Div., N. A. Discharged June 30, 1919. Const. Q. M. at Camp Jackson, Camp Gordon and Ft. Snelling.

WOLFF, HANS HERMANN
Entered service Aug. 15, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 12, 1918. C. O., 556th Engrs., Camp Humphreys; with 553d Engrs., Camp Humphreys.

WOLFF, REINHOLD BERTRAM
Entered service Aug. 13, 1918, as Capt., Engrs., U. S. A. Discharged Dec. 13, 1918. Co. Comdr., 556th Engrs., Camp Humphreys.

WONDRIES, CHARLES HENRY
Entered service Oct., 1918, as 1st Lt., Engrs., U. S. A. Discharged Dec., 1918. With 428th Engrs.

WOOD, BENJAMIN RUSSELL
Entered service Dec. 26, 1917; Capt., Engrs., N. A., 1918; Maj., Engrs., U. S. A., Oct. 10, 1918. Overseas service Feb. 7, 1918–July 29, 1919. Discharged Oct. 27, 1919. In chg.

constr., Gievres, France, Intermediate Storage Depot; in chg. constr., Le Mans, France; supt. constr. Pershing Stadium.

WOOD, FREDERIC JAMES
Entered service Oct. 18, 1917; Maj., E. O. R. C., Feb. 16, 1917. Discharged Sept. 28, 1919. Engr. Officer, Curtis Bay Ord. Depot; in chg. estimates and costs, Ord. Sec., Constr. Div.; Superv. Acquisition Officer on purchase of army camp sites.

WOODARD, WILKIE
Entered service May 8, 1917; Capt., E. O. R. C., Apr. 16, 1917. Overseas service Jan. 29, 1918–July 5, 1919. Discharged July 8, 1919. With 316th and 35th Engrs.; supt. tracks, car shops, La Rochelle, France; designed and superintended constr. Camp Lusitania, Base Sec. No. 4, A. E. F.; attached to Div. of Constr. and Forestry, A. E. F.; attached to Business Mgr., Transportation Dept., S. O. S., A. E. F.; Engr. of Constr., various embarkation areas.

WOODLE, BERNON TISDALE
Entered service May 15, 1917; Capt., Engrs., E. O. R. C., Aug. 15, 1917. Overseas service Oct. 30, 1918–Feb. 4, 1919. Discharged Feb. 12, 1919. With 305th Engrs, Camp Lee; in chg. Engr. Depot No. 7; Intelligence Officer and C. O., H. Q. Co., Camp Humphreys; attached to Div. Military Eng. and Engr. Supplies, S. O. S., A. E. F.

WOODRUFF, CHARLES WILLIAM
Entered service Sept., 1918, as 1st Lt., Engrs., U. S. A. Discharged Mar., 1919. With 403d Engrs., Camp Humphreys.

WOODRUFF, GLENN BARTON
Entered service Sept. 2, 1917; 1st Lt., E. O. R. C., July 6, 1917. Overseas service June 15, 1918–Feb. 12, 1919. Discharged Feb. 14, 1919. With 32d Engrs. in gen. constr., and Asst. Engr. Supply Officer, Base Sec. No. 2, A. E. F.

WOODWARD, EDWIN CARLTON
Capt., Engrs., U. S. A., A. E. F.*

WOODWARD, FRANK COY
Entered service July 27, 1918, as 1st Lt., Engrs., N. A. Discharged Jan. 5, 1919.

WOOLWORTH, WENDELL HOWARD
Entered service Nov. 30, 1916, as 2d Lt., Inf., U. S. A.; Capt., Inf., U. S. A., Aug. 16, 1917; Maj., Inf., N. A., Aug. 6, 1918. Overseas service June 12, 1917–Nov. 16, 1918. Co. Comdr., 28th Inf.; Brig. Adj., 2d Brig., 1st Div.; Asst. to Chf. of Staff, 3d Div. Three divisional citations. Five stars. Wounded Oct. 7, 1918.

WOOTEN, WILLIAM PRESTON
Entered service June 15, 1894; through all grades in C. of E., U. S. A., to Col., May 10, 1917. Overseas service July 22, 1917–July 25, 1919. C. O., 14th Engrs.; Chf. Engr., 3d Army Corps; Chf. Engr., 3d Army. Distinguished Service Medal. Commander St. Michael and St. George, Great Britain. Six stars.

WRIGHT, EDWARD
Entered service Aug. 23, 1918, as Capt., San. C., U. S. A. Discharged Feb. 11, 1919. Instr. School of San. Eng., Camp Greenleaf; Camp San. Engr., Camp Benning.

WRIGHT, FREDERICK
Cpl., Inf., U. S. A.*

WRIGHT, JESSE BERNARD
Entered service Oct. 25, 1918, as Capt., Engrs., U. S. A. Discharged Feb. 17, 1919. With 428th Engrs., Camp Cody; E. O. T. S., Camp Humphreys.

WRIGHT, JOHN BERTRAM
Entered service May 5, 1917; Capt., Engrs., N. A., Oct. 4, 1917. Discharged Jan. 29, 1919. Co. Comdr., 45th, 520th and 209th Engrs.

WRIGHTSON, WILLIAM DAUGHERTY
Entered service Aug. 8, 1917, as Maj., San C., N. A.; Lt. Col., San. C., N. A., Feb. 13, 1918; Col., San. C., U. S. A., Aug. 23, 1918. Discharged Jan. 31, 1919. Organized and became Chf., San. C., U. S. A.

YATES, SHELDON SMITH
F. A. O. T. C., U. S. A.*

YEO, WILLIAM HERBERT WATT
Entered service Jan. 5, 1918; Capt., E. O. R. C., Oct., 1917. Discharged Oct. 23, 1918. Co. Comdr., 531st Engrs.; 5th Engr. Training and Replacement Regt.; 1st Co., Recruit Bn., 5th Engr. Training and Replacement Regt.; Casual Det. Service Bn., Camp Humphreys.

YEREANCE, ALEXANDER WOODWARD
Entered service May 12, 1917; 2d Lt., E. O. R. C., Jan. 15, 1917; 1st Lt., Engr. R. C., Dec. 31, 1917; Capt., Engrs., U. S. A., Oct. 23, 1918. Overseas service May 25, 1918–June 4, 1919. Discharged June 13, 1919. Co. Comdr., 305th Engrs. Four stars.

YOST, HOWARD McCLYMONDS
Entered service May 8, 1917, as Capt., E. O. R. C.; Maj., Engrs., N. A., July 23, 1918. Asst. to Gen. Purchasing Officer, Gen. Engr. Depot, Washington, D. C.; Asst. Chf. Machinery and Eng. Materials Div., Office of Director of Purchase.

YOUNG, CHARLES CLINTON
Entered service Oct. 18, 1918, as 1st Lt., Engrs., U. S. A. Discharged Feb. 13, 1919. E. O. T. S., Camp Humphreys; Inventory Officer, Salvage Dept., Camp Humphreys.

YOUNG, FREDERICK CHARLES
Entered service Aug. 8, 1917, as Capt., E. O. R. C. Overseas service Aug. 22, 1918–May 2, 1919. Discharged May 30, 1919. Regtl. Adj. and Bn. Comdr., 114th Engrs.; Meuse-Argonne offensive. One star.

YOUNG, GEORGE SAMUEL
Entered service Dec. 28, 1917; Capt., E. O. R. C., Sept. 7, 1917. Overseas service Aug. 11, 1918–July 5, 1919. Discharged Sept. 20, 1919. Co. Comdr., 29th and 604th Engrs.; G–2–C, 2d Army, Executive Officer.

YOUNG, SAMUEL McCAIN
Entered service Dec. 28, 1917; Capt., E. O. R. C., Sept. 26, 1917. Discharged Dec. 11, 1918. With 3d Engr. Training Regt., Camp Lee; Office, Chf. of Engrs.; constr. narrow gauge ry., Camp Humphreys; with 98th Engrs., Camp Leach.

ZINN, GEORGE ARTHUR
Entered service July 1, 1883; through all grades in C. of E., U. S. A., to Col. at declaration of war. Retired Sept. 10, 1919. On duty at Portland, Ore., in chg. river and harbor work and fortification constr.

SUMMARY OF SOCIETY MEMBERSHIP SERVING IN THE ARMIES AND NAVIES OF THE UNITED STATES AND ITS ALLIES

American Army

Major-Generals	12
Brig.-Generals	18
Colonels	75
Lt.-Colonels	81
Majors	320
Captains	621
First Lieutenants	257
Second Lieutenants	86
Master Engineers	9
Master Gunners	1
Sergeants	32
Corporals	8
Privates	24
Students, Training Camps	11
Miscellaneous	2
Total	1557

British Army

Brig.-Generals	3
Lt.-Colonels	7
Majors	16
Captains	14
Lieutenants	13
Second Lieutenants	4
Sergeants	1
Corporals	1
Privates	1
Miscellaneous	2
Total	62

French Army

Commandant (rank not given)	1
Captains	3
Sous-Lieutenant	2
Miscellaneous	2
Total	8

Italian Army

Colonel	1
Total in Armies	1628

Summary for Armies

Major-Generals	12
Brig.-Generals	21
Colonels	76
Lt.-Colonels	88
Majors	336
Commandant (rank not given)	1
Captains	638
Lieutenants	270
Second Lieutenants	92
Master Engineers	9
Master Gunners	1
Sergeants	33
Corporals	9
Privates	25
Students, Training Camps	11
Miscellaneous	6
Total	1628

American Navy

Rear-Admirals	5
Captains	7
Commanders	16
Lt.-Commanders	20
Civil Engineers (rank not given)	1
Lieutenants	35
Lieutenants (Junior Grade)	11
Ensigns	9
Chf. Machinist's Mate	2
Marine	1
Seaman	1
Total	108

RECAPITULATION

Total in Army	1628	
Total in Navy	108	
Total on Roll of Honor		1736
Died in Service		27
Grand Total		1763

MEMOIRS OF DECEASED MEMBERS

ALEXANDER MACKENZIE, Hon. M. Am. Soc. C. E.*

DIED FEBRUARY 23D, 1921.

It is with keen regret that his wide circle of friends have realized the passing away of Major General Alexander Mackenzie, United States Army, *Retired,* Chief of Engineers, United States Army, from January 23d, 1904, to May 25th, 1908.

Alexander Mackenzie, the son of Donald Alexander and Mary Ann (Conner) Mackenzie, was born in Potosi, Wis., on May 25th, 1844.

His early education was acquired at the Platteville, Wis., Academy and the Grammar and High Schools at Dubuque, Iowa. He was appointed to the United States Military Academy from Illinois, entering on September 1st, 1860, and having been graduated on June 13th, 1864, a sufficient time before the close of the Civil War to permit him to render gallant and meritorious service—especially in the Department of Arkansas—for which he was brevetted Captain.

Commissioned a First Lieutenant in the Corps of Engineers on his graduation, he was successively promoted to be Captain on March 7th, 1867; Major, April 5th, 1882; Lieutenant Colonel, February 3d, 1895, and Colonel May 3d, 1901. As previously noted, he became Chief of Engineers with the rank of Brigadier General on January 23d, 1904, and in accordance with the provision of the law especially applicable to certain officers who had Civil War service, he was promoted to the rank of Major General on his retirement, on May 25th, 1908.

During his forty-four years of active service, General Mackenzie's experience was naturally a broad one. He first served on the Lower Mississippi River, and thereafter for a brief period on the Great Lakes; following this, from 1866 to 1874, he commanded a company of engineer troops at Willet's Point, N. Y. From June, 1874, to the time of his retirement, his entire service was either in connection with works of river and harbor improvement, or in the Office of the Chief of Engineers at Washington, D. C. Although for about six years, between 1874 and 1880, he was connected, in turn, with the improvement of the Ohio River and with the construction of harbor works and lighthouses on the Great Lakes, it was the period between 1880 and 1895 that was the most notable in his career as an engineer engaged in actual construction.

Between 1880 and 1895, General Mackenzie was in charge of the improvement of the Mississippi River between St. Paul, Minn., and the mouth of the Missouri River, and during these fifteen years the work, which at the time he assumed charge of it was in an elementary and formative stage, developed into what, in all probability, is the most successful example of river regulation known to engineers. River regulation is probably as difficult a problem as any with which engineers have to cope; the variable factors are numerous and are sometimes hard to identify; their influence is frequently obscure and dependent on circumstances and conditions that may originate at points far distant from that which is under treatment; no two seasons are at all alike; and, all told, the conversion of a seemingly lawless, alluvial

* Memoir prepared by Charles Keller, Col., Corps of Engrs., U. S. A., M. Am. Soc. C. E.

stream into one that shall have a smooth, regular channel flowing in gentle curves with adequate depth is a problem the solution of which has usually defied the efforts and patience of engineers of all climes and countries. General Mackenzie's mind, however, was peculiarly adapted to the handling of a problem of this kind, and under his careful and patient direction there came into being standardized plans and methods for treating this stream which have left it to-day, although the project is even now not entirely completed, with a channel of enormously greater capacity for navigation than it had in 1880, when operations under General Mackenzie were begun.

Within the limits of this memoir it would be superfluous to attempt to describe in detail the methods pursued by General Mackenzie, but it is fitting to say that when he came to this work on the Upper Mississippi River it virtually had no useful channel at low-water stages. The channel which existed permitted navigation with a draft not exceeding 1½ ft. as far as La Crosse, Wis., while above that point nothing was possible during low-water periods in the way of navigating the river on a practical scale, although, of course, logs and rafts could be floated. To-day the river is navigable to the Falls of St. Anthony for boats which at extreme low water can always safely draw as much as 4 ft., while at ordinary low water considerably deeper channels may be counted on.

It was while he was on duty on the Upper Mississippi River that General Mackenzie became best known among engineers and to the people of the Central Valleys, where his kindly, congenial manner, his patient consideration for others, and his shrewd, practical judgment won him the confidence, respect, and affection of all who knew him.

In June, 1895, he was called to Washington to become the Principal Assistant in the Office of the Chief of Engineers, where he remained in charge of all matters relating to river and harbor improvements until January 23d, 1904, when he was promoted to be Chief of Engineers. During his service in Washington, General Mackenzie was brought into intimate contact with members of Congress, members of the Cabinet, and with Presidents, all of whom learned to admire him and to respect his judgment and integrity. On his retirement on May 25th, 1908, he took up his residence in Washington and remained there without active occupation until he was recalled to active duty on May 11th, 1917. At that time, at the age of 73, he was still physically quite active, although no longer very strong, and he responded cheerfully to the call of the emergency.

With singular appropriateness, his duty took him back to the Upper Mississippi River where he had spent the happiest and most fruitful years of his service, and where he was hailed with joy by the large number of old friends who still survived. He continued on active service for two years, until 1919, but he found the strain of the work greater than he had anticipated, and it was during this time, no doubt, that the seeds of the organic disease that led to his sudden death, were laid. He was, however, singularly devoted to the discharge of his duty and determined to do everything that one man could do to help in carrying the burden of the Nation, so that, in spite of several warnings which other men would have taken more seriously, he persisted in remaining on duty when it was evident that his health was scarcely adequate to the task. On his return to Washington, his many friends were forced with regret to admit that he had failed noticeably during his

absence, but his sudden death on February 23d, 1921, was a shock as well as a great grief to them all.

It is difficult to summarize in a few words the character of any man, but it seems fitting to say that General Mackenzie was distinguished among men by unaffected simplicity and kindliness of manner, by approachability, and by unassuming but hard-headed common sense that enabled him early in his career to be a successful executive and later in life to be, first, a shrewd and far-sighted assistant and, later, a Chief of Engineers whose career was marked by the success with which he handled the disagreeable and difficult problems of administration constantly besetting the head of a busy Governmental bureau. With his passing the world has lost a most lovable character and the engineers of the country a wise and generous friend and adviser.

General Mackenzie was elected a Member of the American Society of Civil Engineers on February 2d, 1887, and an Honorary Member on May 12th, 1905.

CHARLES ALDO ALDERMAN, M. Am. Soc. C. E.*

DIED MARCH 11TH, 1920.

Charles Aldo Alderman was born in Spring Brook, Wis., on May 21st, 1865. He received his preparatory education at the Eau Claire, Wis., High School and studied Civil Engineering at the Wisconsin State University from 1883 to 1885.

In 1885 and part of 1886, Mr. Alderman was employed as Draftsman and Rodman in the City Engineer's Office at Eau Claire, Wis., and from September to December, 1886, he served as Transitman and Leveler on the construction of the Fairchild and Mississippi River Railway (afterward a part of the Chicago, St. Paul, Minneapolis, and Omaha Railway).

In 1887, he made a survey and map of the Gogebic Iron Mining Range in Northern Wisconsin and Michigan and was also employed as Draftsman and Topographer by the Duluth, South Shore, and Atlantic Railway Company.

In 1888, Mr. Alderman joined the Engineer Corps of the Southern Pacific Railway Company as Draftsman and Topographer on location, and was afterward appointed Assistant Engineer with the Nevada and California Railway Company. He was engaged for a short period in 1889 as Engineer with the San Francisco and North Pacific Railway Company, platting a 900-acre tract of land in Sonoma County, California.

In July, 1889, Mr. Alderman returned to Eau Claire, Wis., as Assistant to the City Engineer, in charge of sewer and street work until April, 1890, when he was elected to the office of City Engineer which he held until 1899. During his term as City Engineer of Eau Claire, Mr. Alderman was also engaged as follows: In 1894, he made a survey of the Chippewa River; in 1895, he acted as Engineer to the Wisconsin State Fish Commission at Bayfield, Wis.; and in 1897 and 1898, he was Chief Engineer on the construction of the Chippewa Valley Electric Railway.

In March, 1899, Mr. Alderman was appointed Chief Engineer of the Great Northern Construction Company, for which company he had entire charge of the construction of various traction lines in Ohio including the Dayton, Springfield, and Urbana Railway, the Columbus, London, and Springfield Railway, the Columbus, Buckeye Lake, and Newark Traction Company's lines, the Columbus, Grove City, and Southwestern Railway, the Central Market Street Railway, at Columbus, Ohio, etc., until 1907, when he entered the employ of J. G. White and Company of New York City, with which firm he remained until 1908.

In 1909, Mr. Alderman was appointed Chief Engineer of the Buffalo and Lake Erie Traction Company and served as such until the completion of the line in 1911, from which time until 1913, he was employed as Assistant Chief Engineer for the Wyandotte Construction Company of Kansas City, Mo., on the construction of the Kansas City and St. Joe lines.

In 1913, he was engaged as Chief Engineer of the Montreal and Bad River Improvement Company, the Ashland Light, River, and Street Railway Company, the Ironwood and Bessemer Railway and Light Company, and the Ironwood Water-Works Company, with headquarters at Ironwood, Mich.

* Memoir compiled from information on file at the Headquarters of the Society.

In 1914, Mr. Alderman opened an office at Ashland, Wis., and engaged in private practice as a Civil and Constructing Engineer, and, in 1915, he became a member of the firm of Diehl and Alderman, Consulting Engineers, of Buffalo, N. Y., which connection he held until his sudden death at his home in Lancaster, N. Y., on March 11th, 1920.

In 1895, Mr. Alderman was married to Emily Belle Farr, of Eau Claire, Wis., who, with one daughter, Louise, survives him. He was a member of the Presbyterian Church, and, in 1894, he became a Master Mason. His professional affiliations included membership in the American Society of Municipal Improvements. He also belonged to the Phi Delta Theta Fraternity.

Mr. Alderman's life was one of untiring energy in his work; he was a real lover of his profession. In business and community life, he gained the unfailing respect and admiration of his fellow-men by his true Americanism. To his family he revealed a depth of character which outsiders could not know, of unselfishness and devotion. By sincere and unostentatious living, he had won a place which words are unnecessary to enhance. His best was his standard always, a standard which mounted continually. To his friends and his family, his life seemed to be complete in the work which he had to do.

Mr. Alderman was elected an Associate Member of the American Society of Civil Engineers on April 6th, 1898, and a Member on April 2d, 1902.

RAFAEL ALVAREZ SALAS, M. Am. Soc. C. E.*

DIED JUNE 29TH, 1920.

Rafael Alvarez Salas was born at Purificacion, Colombia, on November 7th, 1859. When he was quite young his parents moved to Velez, where he first attended school. In order to obtain better educational facilities for him, the family then removed to Bogota, the capital, where he finally completed the Civil Engineering course at the University of Colombia, and also received a diploma in telegraphy.

From 1880 to 1886, Dr. Alvarez was engaged in private practice, consisting mainly of surveys, bridge projects, aqueducts, etc. In 1881, due to his interest and ability in public affairs, he was elected a member of the Assembly of the State of Tolima, Colombia.

From 1887 to 1890, as Assistant Engineer, he was in charge of the construction of the important bridge over the Sumapaz River and several preliminary surveys for the Girardot Railway. In 1891, he served as Chief of the Third Section of these surveys, after which he had charge of the construction of the Agua de Dios Aqueduct. Following this work, he was commissioned by the Colombian Government to make studies of various islands off the Pacific and Atlantic Coasts preparatory to the establishment of a Leper Colony.

In 1892, Dr. Alvarez erected the Portillo Bridge, and in the following year was engaged on the construction of the Girardot Railway as Assistant Engineer. As Chief Engineer, he was then employed on the construction of the Pavas Section of the Antioquia Railway, but relinquished this work on account of illness.

During the period between 1895 and 1900, Dr. Alvarez was twice Superintendent of the Girardot Railway, and also constructed several electric installations on coffee plantations. In 1901, he resumed his private practice, in which year he was elected Corresponding Member of the Instituto de Ingenieros Civiles de Chile. The following year was spent as Principal Assistant Engineer on the boundary survey between Colombia and Venezuela, and when this was completed, he became Chief Engineer of another section, exploring and surveying the Catatumbo, Sardinata, and Tarra Rivers.

In 1902 he was appointed a Founder Member of the Sociedad Nacional de Geografia de Colombia, and in the two years following was again engaged in private practice, his work consisting mainly of electric installations on coffee plantations.

From 1906 to 1907, Dr. Alvarez again turned his attention to railway work as Chief Engineer on Location for the railway from Palmira to Cartago, a branch of the Pacific Railway, about 150 km. in length. This work was followed by an appointment as Director of Construction of the Pacific Railway, in which position he remained until 1919, having in the meantime constructed, as Director General, about 180 km. of the line across the Western Andes and in the Valley of the Cauca. This work is a monument of his skill and resourcefulness as an engineer.

In 1915 Dr. Alvarez was elected to the Senate of Colombia, and in the same year, at the invitation of the Carnegie Institute, he attended the Second Pan-American Scientific Congress at Washington, D. C. He was elected President of the

* Memoir prepared by Luis L. Guerrero, Esq., and Julio Fajardo, Assoc. M. Am. Soc. C. E.

Sociedad Colombiana de Ingenieros in 1920, and almost immediately afterward was sent on a special mission to the United States by the Colombian Government. A few days after his arrival in New York City, he contracted pneumonia which terminated fatally on June 29th, 1920.

By reason of his work and conduct, Dr. Alvarez had always occupied a high position in his own country and abroad, and on his death, the Colombian Congress, the Executive, and many private corporations passed resolutions commemorating his life as a model of good and progressive citizenship.

Dr. Alvarez was elected a Member of the American Society of Civil Engineers on January 4th, 1910.

WILLIAM CREELMAN AMBROSE, M. Am. Soc. C. E.*

DIED JANUARY 4TH, 1909.

William Creelman Ambrose, the son of Michael Head Ambrose and Eliza Rogers Creelman Ambrose, was born at Princeport, Nova Scotia, on May 1st, 1857. After receiving his preliminary education, he taught school for a short period and made a voyage to Antwerp, Belgium, in a sailing vessel. In October, 1875, he entered Kings College, at Windsor, Nova Scotia, from which he was graduated in 1878, with the degree of Bachelor of Engineering (First Class).

After his graduation, Mr. Ambrose went to California, and from December, 1880, to November, 1882, he was employed as Assistant Resident Engineer on the Southern Pacific Railroad, between Lathrop and Los Angeles. In November, 1882, he was appointed Resident Engineer, and, in addition to his duties on the previously named Division, he was in responsible charge of track maintenance, betterment, etc., on the Yosemite and Ventura Branches. In this position his work extended from Lathrop, Cal., to Needles, Ariz., with headquarters at Tulare, Cal., until 1894, when they were moved to Bakersfield, Cal.

Mr. Ambrose continued with the Southern Pacific Railroad as Resident Engineer until 1901, when he went to Fresno, Cal., to take charge of the construction and management of the City and Suburban Electric Car System.

On the completion of this work in 1903, he moved to Berkeley, Cal., and was employed by the Western Pacific Railway Company on the location of its new line along the Humboldt River, in Nevada, and in the Feather River Canyon, in California. During 1904, he was engaged as Chief of Construction on the new railroad over the Nevada Desert to the mining town of Tonopah. In addition to his railroad work, Mr. Ambrose was frequently engaged as a Consulting Engineer in railroad matters.

In the spring of 1905, his health began to fail, and until his death on January 4th, 1909, he was not engaged in the active practice of his profession. He was survived by his widow, Olla Wright Ambrose, and five children.

Mr. Ambrose was always actively engaged in civic and church affairs, and had served several terms as School Trustee and as a Vestryman of the Protestant Episcopal Church. He was widely read in history and English literature, spoke French and German, and was especially fond of music.

Mr. Ambrose was elected a Member of the American Society of Civil Engineers on April 4th, 1888.

* Memoir prepared from information on file at the Headquarters of the Society.

PERCY HERBERT ASHMEAD, M. Am. Soc. C. E.*

DIED NOVEMBER 11TH, 1919.

Percy Herbert Ashmead was born in Philadelphia, Pa., on March 21st, 1867. At the age of 20 he entered the employ of John S. and S. M. Garrigues, serving as Chainman, Rodman, Instrumentman, and Assistant Engineer, until May, 1892, when he became a member of the firm of S. M. Garrigues and Company, of Bryn Mawr and Philadelphia, Pa., which firm was engaged in general engineering practice.

In May, 1893, Mr. Ashmead entered the Engineering Department of the Sanitary District of Chicago, and was engaged on the construction of the Drainage Canal until December, 1896. It was while he was employed on this work that one of the writers, Mr. H. F. Dose, was first associated with him; this association afterward grew into a sincere friendship.

From March to September, 1897, Mr. Ashmead was engaged on location surveys for an electric railway and as Engineer for the contractor on the Dismal Swamp Canal. During the investigations carried on under the direction of the United States Board of Engineers on Deep Waterways, he was in charge, from September, 1897, to October, 1898, of a topographical survey party on the section from Lake Erie to the Hudson and St. Lawrence Rivers.

In 1898, Mr. Ashmead went to Ecuador as Division Engineer on the Guayaquil and Quito Railway, with headquarters at Huigra. He remained in this position until June, 1901, when he went to Cuba as Division Engineer on the Central Division of the Railway of the Cuba Company.

From April to December, 1902, he was connected with the American China Development Company on the location and construction of the proposed railway between Canton and Hankow, China, thus becoming one of the pioneer American engineers in that country. He returned to the United States in 1903 and for a year or more was interested in various railway projects at home. In 1905, after having served as Chief of the Technical Department of the American China Development Company, with headquarters in New York City, Mr. Ashmead returned to China as Engineer-in-Chief of the Canton-Hankow Railway, with headquarters at Canton.

At the conclusion of this work in April, 1906, he returned to New York City and became Secretary and Consulting Engineer of the Philippine Railway Company, which at that time was actively engaged in railroad construction in the Provinces of Panay, Negros, and Cebu, Philippine Islands.

From 1908 to 1910, Mr. Ashmead was Chief Engineer of the Madeira-Mamoré Railway in Brazil, but he was obliged to relinquish his position in the field, on account of failing health, and return to New York City. The Madeira-Mamoré Railway which was completed in 1913, is situated in the Valley of the Amazon, about 1 600 miles from the mouth of the river, and was begun in 1878. On account of ravages among the workers from tropical diseases and other causes which retarded construction, the project was soon abandoned to be revived some thirty years later, when Mr. Ashmead, as Chief Engineer, was appointed to direct the construction of this difficult work.

* Memoir prepared by H. F. Dose and F. Lavis, Members, Am. Soc. C. E.

From 1910 until his death, Mr. Ashmead was engaged in private practice as a Consulting Engineer in New York City. During this period, he served on commissions to investigate disputed boundary questions between Costa Rica and Panama in 1911-12, and between Guatemala and Honduras in 1919. His work in connection with both these missions was most highly commended by the Governments interested, and by the American Geographical Society and the United States Department of State, by which he had been retained.

After the entrance of the United States into the World War, Mr. Ashmead was commissioned, on January 30th, 1918, a Major in the Engineer Officers Reserve Corps and ordered to report to the Chief of Engineers at Washington, D. C., for duty. He served in the office of the Director General of Military Railways until October, 1918, when he was ordered to Camp A. A. Humphreys, Va., as a student at the Engineer Officers Training Camp, and, later, he also served as Unit Discharge Officer of the 5th Engineer Training Regiment.

After his honorable discharge from the Army on December 12th, 1918, Major Ashmead returned to New York City and resumed his practice as a Consulting Engineer. During the epidemic of influenza in the fall of 1919, he was stricken with the disease which, in his case, developed into pneumonia, and after an illness of less than a week he died on November 11th, 1919, at St. Luke's Hospital in New York City. The fatal nature of his illness was thought to have been induced by the undermining of his general health from his unsparing devotion to his work in tropical countries, under the most trying conditions.

One of the writers, Mr. H. F. Dose, had been associated with Major Ashmead professionally both in the United States and abroad, and it is with a sense of pleasure and gratitude that he is able to state that above everything else Major Ashmead considered the welfare of his employees first. Not only did this apply to personal comforts, but also to just and equitable compensation for services rendered and for such amusements and diversions as he could provide for them in regions distant and unexplored, where his profession frequently called him.

No estimate of the professional and personal worth of Major Ashmead can be complete without recognition of his valuable services both to his employers, to his country, and to all engineers, which he rendered in connection with his foreign service. The United States is probably now entering on an era of much greater interest and activity in foreign affairs, one in which American engineers will find many opportunities for professional service abroad. As one of the pioneers in this service, Major Ashmead established precedents and high ideals of life which will be of inestimable benefit and may well serve as examples for those who follow.

Not only was he an engineer of great ability in his special line, but he was held in the highest esteem by all who knew him for his sterling qualities as a man and his lovableness and loyalty as a friend. Major Ashmead was a gentleman in the best sense of the word and succeeded in the foreign field not only by reason of his recognized ability as an engineer, but also by his ability to inspire and preserve lasting friendships among all with whom he came in contact. By his death the Profession has lost a valuable member and the world a man.

He was a member of the Western Society of Engineers and a Fellow of the Royal Geographical Society of Great Britain.

Major Ashmead was elected a Member of the American Society of Civil Engineers on March 6th, 1907.

ROBERT BUNKER COLEMAN BEMENT, M. Am. Soc. C. E.*

DIED MAY 10TH, 1920.

Robert Bunker Coleman Bement, the son of William and Catharine B. (Lewis) Bement, was born in Bacon Hill, Saratoga County, N. Y., on July 3d, 1847. He spent his boyhood days on Staten Island and began his college education in New York University. He remained there until the middle of his Sophomore year, when he entered Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in June, 1869, as a Civil Engineer.

Mr. Bement began his professional work in July, 1869, on the Burlington and Missouri River Railroad in Nebraska, where he remained for two and one-half years, becoming afterward Chief Engineer of the Dubuque and Minnesota Railroad, the Chicago, Clinton, and Dubuque Railroad, and the Iowa Pacific Railroad, with headquarters at Dubuque, Iowa. He built about 135 miles of these roads.

In 1876, Mr. Bement returned to Troy, and became Chief Engineer of the Troy and Boston Railroad shortly after the completion of the Hoosac Tunnel. In 1880, he left railroad engineering and, for two years, was a partner in a firm of steel hammer manufacturers, in Buffalo, N. Y., of which firm he was Manager, leaving to become, in 1882, a member of the wholesale paper firm of Storrs and Bement, of Boston, Mass.

In 1886, Mr. Bement again began the practice of his profession by building the first water-works of Charleston, W. Va., and, in 1887, he continued his hydraulic work by constructing the water-works at West Superior, Wis. At this time, he was residing at St. Paul, Minn., and in 1889, he formed a partnership in that city, known as Robert Bement and Company, building water-works at Bozeman, Mont., and at Lakeside, near Duluth, Minn. Dissolving this partnership, in 1894, he became President of the Board of Water Commissioners of St. Paul, from which position he resigned, in 1899, to become Major of Engineers on the staff of Gen. Wesley Merritt, U. S. A., in the Philippine Islands. He had charge of the Manila Water-Works and also served as Collector of the Port of Manila. On account of an attack of typhoid fever, he was invalided home after remaining in the Islands about a year.

After regaining his health, Mr. Bement resumed the practice of his profession as a Consulting Hydraulic Engineer until 1905, when he became connected with the Robinson and Cary Company, of St. Paul—a railway and mill supplies firm—serving as President and then as Vice-President and Treasurer until August, 1919, when he retired from business.

Mr. Bement died in St. Paul, on May 10th, 1920, and was buried in Oakwood Cemetery, in Troy, N. Y. He had a distinguished career as engineer and business administrator and because of his integrity, great ability, fine presence, and geniality was conspicuous in the commercial, civil, and social affairs of the various cities in which he resided.

He was married on December 31st, 1872, to Mary E. Tracy, daughter of Cornelius L. Tracy, of Troy, N. Y., who, with two children, a son and a daughter, survives him.

* Memoir prepared by Palmer C. Ricketts, M. Am. Soc. C. E.

He was a member of the Minnesota and Informal Clubs of St. Paul, Minn., the University Club of New York City, the Sons of the Revolution, and the Society of Colonial Wars. While a resident of Boston, he was also a member of the Boston Society of Civil Engineers.

Mr. Bement was elected a Member of the American Society of Civil Engineers on September 7th, 1887.

WALTER GILMAN BERG, M. Am. Soc. C. E.*

DIED MAY 12TH, 1908.

Walter Gilman Berg, the son of Albert W. and Helen McGregor (Morse) Berg, was born in New York City, on January 12th, 1858. His father who was well known as a teacher of music, and a composer, was of Italian and German descent, and his mother was of English Puritan ancestry, her great-great-grandfather having been the Rev. Samuel Russell, in whose home Yale College was founded.

When he was nine years old the boy was taken to Europe and was educated in Germany, having been graduated in Civil Engineering from the Royal Polytechnic Institute at Stuttgart, Germany, in 1879. On his graduation, he was awarded the Gold Medal, offered by the King of Wurtemberg, for a treatise on "Spherical Conic Sections", and a scholarship at the Royal Polytechnic Institute.

Returning to the United States in 1879, Mr. Berg entered the employ of the Delaware Bridge Company as Draftsman and Shop Inspector, which position he retained until the spring of 1880, when he was appointed Engineer of Bridges with the Richmond and Alleghany Railroad Company.

In 1882, he entered the service of the East Tennessee, Virginia, and Georgia Railroad Company as Principal Assistant Engineer of the Construction Department.

In January, 1883, Mr. Berg began his service with the Lehigh Valley Railroad Company as Assistant Engineer in charge of the Lehigh and New Jersey Division, on the design and construction of shop buildings, round-houses, surveys for new branches, etc. He also had charge of the office of the Chief Engineer of the road. In addition to these duties, in 1886, he designed and constructed the Lehigh Valley Creosoting Plant at Perth Amboy, N. J. He was put in charge of the operation of this plant which was among the first of its kind in the United States, and through his studies in connection with its work, Mr. Berg became an authority on creosoting processes.

On November 1st, 1887, he was appointed Principal Assistant Engineer of the Lehigh Valley Railroad, with headquarters at Jersey City, N. J. In this capacity he superintended the extension of the road in New Jersey and the development of its water-front properties, having constructed the double-track road from South Plainfield to Jersey City, N. J., and designed and constructed the first piers at the latter place.

On December 1st, 1898, Mr. Berg was made Engineer of Maintenance of Way of the Lehigh Valley Railroad, with headquarters at South Bethlehem, Pa., and, later, was also placed in charge of the Engineering Department. He retained this position until February 1st, 1900 when he was appointed Chief Engineer of the railroad, in charge of construction, continuing as such until his death. Among the works accomplished by Mr. Berg in this position, were the design and construction of the Railway Shop System at Sayre, Pa., including a large reservoir system, grade-crossing elimination in various cities, harbor improvements at Buffalo, N. Y., and New York City, studies and designs for large terminal yards, etc.

* Memoir prepared from information furnished by F. E. Schall, M. Am. Soc. C. E., and on file at the Headquarters of the Society.

He had been appointed as the representative of the American Railway Engineering and Maintenance of Way Association (now the American Railway Engineering Association), at a Conference of Governors called by President Roosevelt and was ready to start for Washington, D. C., when he was seized with an attack of acute indigestion and passed away suddenly at his home in New York City, on May 12th, 1908.

Mr. Berg was married in 1893 to Miss Ruby Burke, of Jennings, Va., who died in November, 1904. He was survived by two sons, and by his mother, sister, and two brothers.

Mr. Berg had always shown great musical ability and although in later life his professional duties prevented him from cultivating his talent to any great extent, he was able to read at sight and play well both the piano and the organ.

As a student, Mr. Berg was exceptionally brilliant as evidenced by the fact that he had read the proof for several German publications, instructed in mathematics, and had been graduated with honors. During his school vacations, he had taken long tours on foot through Germany, Austria, Switzerland, Italy, and France, thus adding greatly to his knowledge and culture. As an engineer, he had wonderful executive ability combined with a breadth of view and a faculty for the closest attention to details. In addition to his duties with the Lehigh Valley Railroad Company, he was frequently called on by other companies and individuals to act in a consulting capacity.

Mr. Berg was of a kindly and genial disposition, always courteous to others and ever ready to give advice and assistance to those in need, and always loyal to his obligations and responsibilities.

He had written many technical and scientific articles and books, among the latter being "Buildings and Structures of American Railroads", "Timber Tests", and "Railway Shop Systems", as well as papers on railroad subjects to be presented before various technical societies.

He was a member and sixth President of the American Railway Engineering Association, to the work of which he had devoted a great deal of his time, Past-President of the Association of Railway Superintendents of Bridges and Buildings, a member of the American Roadmasters' Association, the Eastern Maintenance of Way Association, Railway Superintendents' Association, and the American Society for Testing Materials. He was also a member of the New York Railroad Club, the Founders and Patriots of America, and Holland Lodge, F. and A. M.

Mr. Berg was elected a Member of the American Society of Civil Engineers on February 5th, 1896.

MAXIMILIAN FERDINAND BONZANO, M. Am. Soc. C. E.*

DIED OCTOBER 30TH, 1920.

Maximilian Ferdinand Bonzano was born in Detroit, Mich., on May 13th, 1858, and was the eldest son of the late Adolph Bonzano, M. Am. Soc. C. E., and Laura (Goodell) Bonzano.

His father's family was one of a long line of eminent scientists and engineers. His father in particular was one of the most prominent engineers of his time and a pioneer in the design and fabrication of long-span bridges, having been Vice-President and Chief Engineer of the Phœnix Bridge Company, the builders of the first elevated railroads in New York City and Brooklyn, and of other notable structures.

Maximilian Ferdinand Bonzano inherited the engineering instincts of the family, and therefore it was natural that he should have selected the Engineering Profession for his life work. He was educated at Rensselaer Polytechnic Institute, entering with the class of 1878, and spent his summer vacations during his collegiate career in machine shops and with the bridge-erecting forces then engaged in the erection of some of the long-span bridges of that day. He became an experienced mechanic, and the practical experience picked up at this time was most helpful to him during his later career when he was the operating officer of several railroads. It enabled him to give practical attention to the mechanical departments and his suggestions to the officials of these departments were most helpful.

Mr. Bonzano entered railroad service on the Philadelphia, Wilmington and Baltimore Railroad in June, 1878, as Assistant Engineer. When that road passed to the control of the Pennsylvania Railroad, he was transferred and placed in charge of extensive terminal improvements in and about Philadelphia, Pa. In 1882, he was transferred to the General Superintendent's Office at Altoona, Pa., and in October of that year he resigned from the service of the Pennsylvania Railroad to become an Assistant Engineer on the Philadelphia and Reading Railroad. In April, 1883, he was appointed Roadmaster of the New York Division, and, in 1884, became Division Engineer of that road.

In 1886, Mr. Bonzano was appointed Superintendent of the North Penn and Bound Brook Division and the Germantown and Norristown Branch of the Philadelphia and Reading Railroad, and, in 1889, he was made Assistant General Superintendent of the System. He became General Superintendent in 1891, and served as such until 1893, when he became Assistant to the Vice-President, the late Robert H. Sayre. This was during the period when the Reading Railroad had taken over the Lehigh Valley, the Central Railroad of New Jersey, and the Central New England Railroads. In this office he had immediate charge of all engineering matters, including maintenance of way and construction. When the road passed into the hands of receivers and the Lehigh Valley and Central New England Railroads were dropped from the Reading System, the offices of Vice-President and General Manager were abolished. The former General Manager became General Superintendent, and Mr. Bonzano decided to leave the service of the Reading Railroad in June, 1893.

* Memoir prepared by F. A. Molitor, M. Am. Soc. C. E.

It was during the eleven years of Mr. Bonzano's connection with the Reading Railroad that his professional and engineering instincts made a lasting impression on that road. When Division Engineer he was one of the first to use reinforced concrete extensively in bridge and culvert details, and many problems of foundations in the reconstruction of the substructures of the railroad were boldly and successfully met. Many of these structures now stand as monuments to Mr. Bonzano's broad grasp of engineering details, his courage in design, and his genius in execution. He gave much time and thought to the improvement of track details and was one of the pioneers in the use of heavy rails and refinement in track details, especially in frogs and switches, and in the better and more scientific use of stone ballast and automatic signaling. It was during this period, also, that he invented and patented the steel tie which still bears his name. When his responsibilities were broadened by taking charge of operation, his scientific use of statistics, and analyses of transportation factors were brought to bear on the many operating problems of this great coal-carrying railroad. It was here, also, that he gained the broadest experience in handling heavy passenger and freight traffic.

Between 1894 and 1896, Mr. Bonzano was General Manager of the South Jersey Railroad, and between 1896 and 1897 he was General Manager of the Chattanooga Southern Railroad, then owned and controlled by the late Russell Sage, for whom he had become expert adviser in respect to his many railroad interests. In 1897, Mr. Bonzano went with the Columbus, Sandusky and Hocking Railroad, as General Agent for the Receiver, S. M. Felton, M. Am. Soc. C. E. As such, he had entire charge of the property for the Receiver. In 1898 he returned to the Chattanooga Southern Railroad, and remained until 1900, when he was appointed Chief Engineer of the Toledo, St. Louis and Western Railroad. As such, he handled the proposed rehabilitation of this property.

In 1902, he became Chief Engineer of the Pittsburgh, Shawmut and Northern Railroad, in charge of construction, remaining until the completion of this road's construction programme in 1904. In 1905 and 1907, he was General Manager of the Chapman Coal and Iron Company, of Virginia.

Between 1907 and 1911, Mr. Bonzano was in private consulting practice, giving particular attention to mining and railroad properties and was Consultant on the operating features of many mines in the Southeastern District.

In 1912, he assisted W. J. Wilgus, M. Am. Soc. C. E., in the valuation of the Lehigh Valley Railroad. This Company was the first to recognize the necessity of an individual valuation of its property, and many experienced and prominent railroad engineering experts were on Mr. Wilgus' staff.

On the death of his father, Adolph Bonzano, in May, 1913, Mr. Bonzano retired from practice and spent his remaining years on his estate in Connecticut.

He was married, in 1880, to Mary Geary, the daughter of ex-Governor John M. Geary, of Pennsylvania, and one of the famous generals of the Civil War. Mr. Bonzano died of heart disease on October 30th, 1920, leaving a widow and two daughters, Mary, the wife of Col. John E. Harris, U. S. A., and Laura, unmarried.

Although Mr. Bonzano's entire professional career was connected with railroads where the personal element is often lacking, and where engineering design and construction standards are generally credited to the railroad property, never-

theless, his many associates have recognized his influence in the advance of railroad engineering details. It is known to many that he was a master in the economics of railroad location, that his design of construction details expressed advanced thought and his executions were economic in the greatest degree. It was his personal interest and pleasure to use his thought and energy on railroad properties of lesser financial standing than on those with a large treasury. The engineering skill that he displayed on the railroads with which he was connected invariably showed ample interest on each dollar invested.

Mr. Bonzano was particularly interested in the younger engineers and gave them encouragement, advice, and support. He was ever ready to give them his time and aid, to assist them to employment, and he followed their future careers with interest. Many engineers of the generation succeeding him owe their start in life to his interest in them, and their later success to his influence and support. He was a splendid judge of men, and he never hesitated to back his judgment in securing them employment and advancement.

Although many of his associates and acquaintances of his own generation have passed away, those remaining recognize his talents and those of the succeeding generation whom he has helped by practical encouragement, revere his memory. Mr. Bonzano was the most striking example of a man strong in his convictions and belief and ever ready to support them. He was unbending in his support of the ethics of the Profession, of efficient administration of public and private affairs, and was, in a word, a sterling American.

Mr. Bonzano was a member of the Union League Club of Philadelphia, Pa. He was elected a Member of the American Society of Civil Engineers on January 6th, 1886.

WILLIAM HENRY BRENTON, M. Am. Soc. C. E.*

DIED FEBRUARY 11TH, 1921.

William Henry Brenton was born in Petersburg, Ind., on May 3d, 1861, his parents, Ellen Brenton and Peter I. Brenton, having been pioneer farmers in Southern Indiana. His early education was received in the public schools of Petersburg, while his technical education was acquired at the University of Michigan, from which he was graduated in 1883 with degree of B. S. in Civil Engineering.

After his graduation, Mr. Brenton entered the employ of the Chicago and Alton Railroad Company on construction in Missouri, and there began a career of responsible service, in connection with the building and maintaining of railroads in the West that has been equalled by only a few engineers. From 1886 to 1888, he was Division and Resident Engineer on extensions of the Chicago, Rock Island and Pacific Railway in Kansas, Nebraska, and Colorado, and from 1889 to 1891 he was Assistant Engineer for the Colorado Midland Railroad Company, and built many difficult and important tunnels and bridges for that Company through the Rocky Mountains of Colorado. In 1892 and 1893, he was again with the Chicago, Rock Island and Pacific Railway Company in Nebraska, Indian Territory, and Texas, as Assistant Chief Engineer of construction, and built many important lines.

In 1894, 1895, and 1896, Mr. Brenton served the Rock Island Company in Oklahoma and Indian Territory as Roadmaster, and, from 1897 to 1900, he was Roadmaster and Engineer for the Colorado Midland Railway Company. From 1901 to 1903 he was again employed by the Rock Island Company as Division Engineer in charge of some of the heavy construction on the El Paso Extension through New Mexico. In 1906 he supervised important terminal revisions for the Atlanta, Birmingham, and Atlantic Railway Company in Georgia.

Mr. Brenton was Assistant City Engineer of Colorado Springs, Colo., in 1905. He left this work to become Engineer for the Flick Construction Company and had charge of a number of important projects, among which were the grading of the four-track electric line of the New York, Westchester and Boston Railway; the New York Central Yards in Buffalo, N. Y.; and the double-track line of the Atchison, Topeka, and Santa Fe Railway in Missouri.

In 1909, Mr. Brenton decided to give up the active practice of engineering and moved to Eugene, Ore., in order that his children might have the educational advantages of the University at that place. His previous life, however, had been too active and he chafed under the restrictions of retirement. Because of his broad and extensive knowledge of all kinds of construction work, his services as Consulting Engineer were soon in demand by railroads and contractors in various parts of the United States.

When the Federal Valuation Act was passed by Congress in 1913, Mr. Brenton decided to place his experience and knowledge of the railroad systems of the West at the service of the Government. He took the examination for Senior Civil Engineer and ranked among those having the highest grading. As soon as the

* Memoir prepared by R. A. Thompson, M. Am. Soc. C. E.

work was organized, he was appointed Field Engineer for the Pacific District, with headquarters at San Francisco, Cal., and was placed in charge of a number of roadway and track field parties. He served in this capacity from July 1st, 1914, to March 1st, 1920, and during that period inventoried about 20 000 miles of railroad for valuation purposes. Thereafter, he was engaged in adjusting inventories as Assistant Engineer of the Northern Pacific Company, until his appointment on September 18th, 1920, as Senior Civil Engineer in charge of the Pacific District Branch Office at San Francisco, in which capacity he had the supervision of improvement reports prepared by railroads for the purpose of maintaining their valuations. It was on one of the first assignments in this new position that he was taken seriously ill at Jerome, Ariz., and died at the United Verde Hospital in that city on February 11th, 1921.

Mr. Brenton was a man of high ideals, of pleasing personality, and a most agreeable associate. He was an energetic and untiring worker, and although of a quiet and retiring disposition, he made many firm friends who deeply regret his loss. His acquaintance was particularly broad and extensive among engineers and railroad officials throughout the West, and his reputation for integrity and fair-mindedness and his sound and unerring judgment in engineering matters gave him the highest standing in his Profession. He was a true and sincere friend, a generous and loving husband and father, and a respected and honored citizen.

In 1885, he was married to Miss Mary Foster, of New York City, who, with four children, survives him.

Mr. Brenton was elected a Member of the American Society of Civil Engineers on April 18th, 1916.

CHARLES BENJAMIN BRUSH, M. Am. Soc. C. E.*

DIED JUNE 3D, 1897.

Charles Benjamin Brush was born in New York City on February 15th, 1848, one of three sons and three daughters of Eliza (Turck) and Jonathan E. Brush. He was graduated from New York University in 1867 with the degree of Civil Engineer, and returned to that institution as Instructor in 1874. He was promoted to the position of Professor of Civil Engineering in 1888, and became Dean of the School of Engineering in 1895, when he was honored with the degree of Doctor of Science. Although his health failed about this time, he continued officially at the head of the Engineering School until his death.

Immediately after graduation, Mr. Brush was connected with the Engineer Corps on the Croton Aqueduct, New York City, for two years, but in 1869 he formed a partnership with the late Arthur Spielmann, M. Am. Soc. C. E., at Hoboken, N. J. This firm engaged in extensive engineering work for the following: The Hoboken Land and Improvement Company; the Hoboken Ferry Company; the Hackensack Water Company; the North Hudson County Railway Company, and for many other corporations and communities in Bergen and Hudson Counties, New Jersey.

Mr. Brush also was connected with work on the first tunnel under the Hudson River, acted as Assistant Engineer for the proposed New York-New Jersey Bridge over the Hudson River, and as Engineer for the Contractor who built the piers for the 181st Street Bridge over the Harlem River. He was Consultant on the reconstruction of the famous ceiling of the Senate Chamber at the Capitol, Albany, N. Y. As Chief Engineer, Consultant, or Managing Director, he either built, remodeled, or enlarged water and sewerage systems in New Rochelle, Irvington, Highland Falls, Southampton, Far Rockaway, and Syracuse in New York State; Plainfield, and the great Hackensack System, in New Jersey; Lancaster and Easton, Pa.; Alliance, Ohio; Kansas City, Mo.; Kansas City, Kans.; and Portsmouth, Suffolk, and Berkeley, Va.

Mr. Brush also organized branches in his office for the preparation of insurance atlases, and for making boring tests for foundations of buildings or bridges. After the death of Mr. Spielmann in 1883 and until his own health failed, he conducted the very extensive practice himself, but about 1895 he took into partnership his senior assistant, W. F. Whittemore, M. Am. Soc. C. E., under the firm name of Charles B. Brush and Company. The firm continued to conduct business until the death of Mr. Brush in 1897, and for some years following, until the virtual retirement of Col. Whittemore from active practice.

After the lapse of nearly twenty-four years it is interesting to note the high standing in professional and civic life attained by those who, as students or assistants, passed under the influence of the inspiring and successful engineer and all-round Christian gentleman, Charles B. Brush. He was an occasional contributor of technical papers, and frequently submitted discussions of remarkable clearness and interest. His reports were models of terse English and convinc-

* Memoir prepared by Louis L. Tribus, M. Am. Soc. C. E.

ing logic. Personal friendship was his dominant characteristic—clients became friends, students became admirers, employees respected and served him loyally. His judgment of men was remarkable, and his ability to assemble and present ideas in intelligent and convincing form was much appreciated by his clients and associates.

Mr. Brush was a member of the Engineers' Club, New York City, the American Water Works Association, and the New York University Alumni Association. He was an active worker in the Central Presbyterian Church of New York City; Superintendent of its Mizpah Chapel Sabbath School; and an Elder for many years. He was a valued Director of the Charity Organization Society.

In 1883, Mr. Brush was married to Carrie F. Cooley, who, with three children, Anna C., Joshua C., and Charles B., Jr., survives him.

Mr. Brush was elected an Associate Member of the American Society of Civil Engineers on September 6th, 1871, and a Member on September 5th, 1877; he served as a Director from 1888 to 1891, and as a Vice-President from 1892 to 1894.

WILLIAM ASHBURNER CATTELL, M. Am. Soc. C. E.*

DIED OCTOBER 10TH, 1920.

William Ashburner Cattell was born on June 16th, 1863, at Princeton, N. J., the son of Anna Ashburner and Thomas Ware Cattell. His boyhood and youthful days were spent in Lincoln, Pa., where his father held an honored position with the University.

In 1880, Mr. Cattell entered the Pardee Scientific Department of Lafayette College, at Easton, Pa., and completed the four-year course there, having been graduated in 1884 with the degree of Civil Engineer. Immediately after his graduation he began the active practice of his profession and during the next five years was engaged in making surveys and valuations of the railroad and canal property in the State for the State Board of Railroad Assessors of New Jersey. During this period he also served the Atchison, Topeka and Santa Fé Railroad Company on preliminary and location surveys in Kansas and Indian Territory.

Returning East in 1889, he served, until 1897, with the Long Island Railroad as Assistant Chief Engineer in charge of the Construction Department, which detail included the practical reconstruction of the road and the design and construction of bridges, buildings, docks, piers, and terminals incident to the reconstruction and extensions.

Severing his connection with the Long Island Railroad in 1897, Mr. Cattell opened an office as Consulting Engineer in New York City and until 1905 was engaged in the general practice of his profession, serving during that time as Consulting Engineer for the Brooklyn Park Department for bridge construction; the Ohio Southern Railroad for bridges and general improvement; and the Manhattan Beach Company for the installation of electric light, power, and refrigerating plant, electric railroad, marine bulkheads, and shore protection.

During this time he was associated with prominent New York engineers in the valuation of many manufacturing plants, and made reports covering important existing and projected railways as well as terminals. He also reported on extensive irrigation projects since constructed by the United States Reclamation Service, and on a number of water supply and power projects in various parts of the United States; prepared estimates for bids for various concerns on extensive improvements, such as the Atlantic Avenue improvements of the Long Island Railroad and that section of the New York Rapid Transit Subway from the City Hall, New York City, to Flatbush Avenue, Brooklyn, including the tunnel section under the East River, as well as for the track elevation of the New York Central Railroad at Schenectady, N. Y.; and assisted in the examination of the new filtration plants, pumping stations, and aqueduct tunnel of the Philadelphia Water-Works.

Many other services were performed by Mr. Cattell while he was engaged in the practice of his profession as Consulting Engineer in New York City, and the foregoing only illustrates the nature and scope of his work up to the time that he became associated with the firm of E. H. Rollins and Sons, of San Francisco, Cal.,

* Memoir prepared by the following Committee of the San Francisco Section of the American Society of Civil Engineers: J. Otis Burrage and J. W. Ferguson, Members, Am. Soc. C. E., F. H. Fowler, Assoc. M. Am. Soc. C. E., and W. D. Durand, Esq.

which firm he served as Consulting Engineer from 1905 to 1908, making examinations and reports, for bond purposes, on various steam and electric railroads, among the latter being the Western Pacific Railroad. It was during these years that he served as President of the Petaluma and Santa Rosa Electric Railway.

From 1908 to 1917, Mr. Cattell was engaged in general practice in San Francisco, with offices in the Foxcroft Building. He made a reconnaissance survey and report on the Valdez-Yukon Railroad project in Alaska, valuations of the Oakland Water-Works System, Oakland, Cal., the Oakland Traction System, and the Los Angeles City Railway System, as well as other valuations covering railways, power, and gas plants. His particular interest in water-front and harbor improvements caused him to be retained by the San Francisco-Oakland Terminal Railways to report on its Terminal Pier with special reference to the effects of salt water on concrete piers, and, later, as Consulting Engineer for the same Company, in connection with the terminal and harbor developments.

It was during these years that he made a reconnaissance survey for the United States Forest Service of a railway line along the Klamath River, California; served as Consulting Engineer for the Peoples Water Company, Oakland, Cal., and for the Los Angeles Railway Corporation, with special reference to valuation matters. He made a valuation of the Honolulu Rapid Transit and Land Company, Honolulu, Hawaii, and a valuation of the properties of the Oahu Railway and Land Company, also in Hawaii.

Perhaps his best known work was in connection with the Committee of Management of the International Engineering Congress, for which he acted as Secretary-Treasurer from March, 1913, to the practical close of the work of the Committee in the fall of 1916. During this period Mr. Cattell gave a large part of his time to the duties devolving on him in this connection, and during 1915-16 he devoted himself almost entirely to this work. Being responsible for the organization and general management of the office of the Congress during these years, he gave his best thought and energy to the manifold requirements of the work, with special reference to all matters connected with general organization, with publicity, and with the collection and handling of funds. The publications of the Congress include a volume of general proceedings, with a history of the organization and development of the Congress from its initial inception to its final distribution of printed volumes of proceedings. This volume was prepared almost entirely by Mr. Cattell and furnishes in clear and condensed form a most interesting and valuable record of this great undertaking. It is not too much to say that the high mark of achievement set by this Congress was due in large degree to the excellent judgment and unremitting zeal with which Mr. Cattell undertook and carried through the duties of Secretary-Treasurer of the Committee of Management of the Congress.

With the first call of the Government for the formation of an Engineer Officers' Reserve Corps early in 1917, Mr. Cattell enrolled and was commissioned a Major of Engineers on March 1st, 1917. This was followed by the entrance of the United States in the World War, and Major Cattell was ordered to report for duty at Camp Lee, Va., on January 5th, 1918.

Early in the summer of 1918 he was transferred from the Training Camp to the Office of the Chief of Engineers, at Washington, D. C., to organize the Section

of Historical and Statistical Data. Under his able guidance the Section collected and put into permanent form all historical and statistical information concerning every unit of engineers, from the date its organization was first proposed in the memoranda of General Pershing or the General Staff, through its authorization, mobilization, training, and shipment overseas, to the date on which it was finally mustered out. The data thus collected should be invaluable as a guide in the organization of American engineering forces should history ever repeat itself.

With the retirement of Major Cattell, on September 30th, 1919, from the United States Army, ill health overtook him, and he did not again take up the active practice of the profession of Civil Engineering. He spent some months in Philadelphia, Pa., and returned to California in January, 1920, where he resided in Alhambra until his death, after a long illness, on October 10th, 1920.

In 1889, Major Cattell was married, at Lincoln, Pa., to Jennie Woodhull. Mrs. Cattell and their three children, Gilbert Woodhull, Anna Ashburner, and Dorothy, who survive him, are now living at Alhambra.

Major Cattell was a member of the Phi Delta Theta Fraternity, and the Masons. Among the professional societies which were honored by his membership were: the American Society of Mechanical Engineers, the American Institute of Consulting Engineers, the Pacific Association of Consulting Engineers, the American Water Works Association, the Institution of Civil Engineers (Great Britain), and the American Railway Engineering Association.

A man of the highest ideals, loving, thoughtful, and generous in his dealings with his fellow men, and an engineer of exceptional ability, the Society, by the death of Major Cattell, has lost one of its most earnest and valued members, one who had served the Society in many ways as a member and as an officer, and who will be long remembered by the membership for his excellent work and remarkable personality.

Major Cattell was elected a Member of the American Society of Civil Engineers on October 7th, 1896, and served as a Director from 1912 to 1914.

ALBERT SAFFORD CHEEVER, M. Am. Soc. C. E.*

DIED FEBRUARY 17TH, 1913.

Albert Safford Cheever, the son of Tracy P. and Louisa (Kilburn) Cheever, was born at Chelsea, Mass., on September 17th, 1857. He received his education in the public schools at Chelsea, and, in 1880, began his life work in the office of the General Superintendent of the Fitchburg Railroad Company.

In January, 1881, Mr. Cheever was transferred to the Engineering Department of the Fitchburg Railroad as Rodman. He was soon promoted to be Chief of a field party, and for the next five years he was engaged in laying out work on track, bridges, etc., on the double-tracking of the road between Ashburnham and Greenfield, Mass.

In October, 1886, he was made First Assistant Engineer of the Fitchburg Railroad, in charge of the office at Fitchburg, Mass.

In May, 1887, when the Fitchburg Railroad Company acquired the Troy and Greenfield, the Troy and Boston, and the Boston, Hoosac Tunnel and Western Railroads, Mr. Cheever was made Division Engineer, with headquarters at North Adams, Mass., in charge of the reconstruction and maintenance of these roads.

In November, 1890, he was appointed Chief Engineer of the Fitchburg Railroad Company, with headquarters at Fitchburg, Mass., which position he retained until August 30th, 1897, when he resigned to engage in manufacturing in Cleveland, Ohio.

During the latter part of 1898 he returned East, and was appointed Assistant to the President of the Fitchburg Railroad and soon afterward to his old position as Chief Engineer. Mr. Cheever remained in this position until July 1st, 1900, when the road was leased by the Boston and Maine Railroad Company, of which Company he was then made Assistant Chief Engineer.

In November, 1902, he was appointed Superintendent of the Fitchburg Division of the Boston and Maine Railroad, and in November, 1912, he was made Assistant to the Vice-President of the road in charge of operations, which position he retained until his death, at his home in Somerville, Mass., on February 16th, 1913, from an attack of acute indigestion.

On June 7th, 1893, Mr. Cheever was married, at Fitchburg, Mass., to Miss Josephine M. Grant, who, with a son and a daughter, survived him.

In a memoir prepared for the Boston Society of Civil Engineers, J. P. Snow and H. W. Hayes, Members, Am. Soc. C. E., have written:

"Mr. Cheever was essentially an all around railroad man with preference for the constructive engineering side. He was always attentive to his duties in whatever direction they might be and was popular with subordinates and superiors alike. While he exacted good work from contractors, he dealt fairly with them and was very successful in handling construction work. He was a perfect gentleman and a genial companion, and his loss is keenly felt by all who knew him."

He was a member of the Boston Society of Civil Engineers and Aurora Lodge F. and A. M., of Fitchburg, Mass.

Mr. Cheever was elected a Member of the American Society of Civil Engineers on June 7th, 1893.

* Memoir prepared from information on file at the Headquarters of the Society.

GEORGE LYON CHRISTIAN, M. Am. Soc. C. E.*

DIED APRIL 25TH, 1920.

George Lyon Christian was born on June 26th, 1863, at Yonkers, N. Y. His parents were John Joseph and Albertine Solsberg (Johnson) Christian. His paternal ancestors were inhabitants of the Isle of Man, one branch having come to America in the early part of the Seventeenth Century and settled in Virginia. Several members of this branch served in the Patriot Army during the American Revolution. His maternal ancestors were of Norse stock from Christiania, Norway, and French from Alsace-Lorraine. These families came to America in the early part of the Nineteenth Century.

Mr. Christian received his early education in the public schools of Yonkers and by private tuition; his technical training at the University of Denver, Cooper Union, New York City, and the Brooklyn Polytechnic Institute; and his early practical engineering training in that excellent school of experience to which many American engineers owe much, railroad work in the Far West.

From 1887 to 1889, Mr. Christian was a member of the Engineer Corps of the Union Pacific Railroad Company in various capacities on preliminary and location surveys in the Rocky and Sierra Nevada Mountains. In the spring of 1888, he was assigned to a party engaged in running a line from Robinson Pass, near Leadville, down the Eagle River to Grand Junction, Colo. From there his party was sent to South Park, Colo., to run a line from Jefferson to Canyon City and through the famous Cripple Creek District. This was before the discovery of the vast mineral deposits in that district. The party was recalled to Denver in December, 1888, re-organized, and with several other parties sent to Redding, Cal., to run a line from Ontario, Ore., to the Sacramento River. The party to which Mr. Christian was assigned started its work at Beckworth Pass about January 1st, 1889, and spent a strenuous six months in the rough country there, working down the Feather River and Pitt River Canyons to Oroville, Cal., where the parties were disbanded, the Union Pacific officials having become convinced, apparently, that the project was not a practical one.

In July, 1889, Mr. Christian was assigned to one of several new parties organized by the Union Pacific Railroad Company and sent to Milford, Utah, his party being in charge of Mr. H. W. Wood. Milford was then the southern terminus of that branch of the Union Pacific which extended south from Salt Lake, and the parties were set to work surveying for an extension of this branch known as the Pioche (Nevada) Extension. The work lay across the Escalanta Desert through the Clover Valley Canyon and down the Meadow Valley Wash. From December, 1889, to November, 1890, Mr. Christian was engaged on the construction of this extension as Instrumentman on the Mountain Division and, later, as Construction Engineer in charge of 15 miles of construction. The failure of Baring Brothers, of London, England, at this time, and the resulting shock to the financial world, caused a sudden stoppage of the construction work in the latter part of 1890, and the engineering force was disbanded.

During November and December, 1890, Mr. Christian was engaged as Draftsman of a locating party of the Denver and Rio Grande Railway Company in the

* Memoir prepared by Robert Ridgway, M. Am. Soc. C. E.

mountains of Colorado. From September to November, 1891, he was employed on the construction of the Silver City and Northern Railroad in New Mexico as Transitman in charge of grading, track-laying, construction of all piling and trestle bridges, and the preparation of final estimates and maps for the entire 18 miles of road which extended from Whitewater through Hanover Gulch to the Santa Rita Mines.

Mr. B. C. Brandstadt, who was associated with him in most of his Western work, writes:

"Christian had many friends in the West. All who knew him there held him in highest regard. * * * In conversation with him one day about this time (1891), he expressed the belief that, contrary to opinion, opportunities in the engineering field were far greater in the East than in the West, and this may be one of the reasons why he quit the West shortly after. I think, however, that a man of his quality and ability would succeed in almost any part of the country. During these years I learned to value his friendship and loved him as a brother. I had the highest regard for him as a man and as an engineer. He was careful, accurate, and indefatigable in all his work, and I often recall the words of our Chief Engineer at the time our party was breaking up at Milford, Utah. He said, speaking of George, 'There is a boy who will make good.'"

In 1892, Mr. Christian came back to his native city and, after a short service with the New York and Boston Railroad and the New York Central and Hudson River Railroad Companies, entered the employ of the late William Henry Baldwin, M. Am. Soc. C. E., City Surveyor and Engineer of the Board of Water Commissioners, of Yonkers, N. Y., where he remained until 1897. He was in charge of Mr. Baldwin's office during his absence, and, under him, of all street and sewer construction, topographical surveys, and water-works surveys and construction, including the Fort Field Distribution Reservoir which had a capacity of 60 000 000 gal., and was constructed at a cost of $250 000. During this period, on April 22d, 1896, he was appointed by the Hon. John F. Peene, then Mayor of Yonkers, as a Civil Service Commissioner, and served the city in this capacity until May 14th, 1898.

On August 2d, 1897, Mr. Christian joined the Engineering Staff of the City of New York, entering as a Draftsman in the Department of Street Improvements of the 23d and 24th Wards. These wards comprised that part of the city which was north of the Harlem River and which became the Borough of the Bronx when the city lines were extended in 1898 to include the five boroughs of the present city. He was made a Transitman in the same Department on October 1st, 1897, and an Assistant Engineer on October 1st, 1900. As Assistant Engineer he was for several years in responsible charge of contract work for much important sewer construction in the Borough of the Bronx.

When the project for a great storm-relief sewer to extend from Webster Avenue to the Harlem River was being considered in 1904, Mr. Christian was placed in charge of the preparation of the plans. This sewer was designed to take care of the storm flow of about 1 300 acres and to relieve the existing Webster Avenue Sewer south of Claremont Parkway. As built, it was about 7 000 ft. long, of which length about 5 800 ft. was in tunnel. It had a horseshoe cross-section, 14 ft. wide and 11 ft. 8 in. high, and was lined with concrete throughout, the open-cut portion of the lining being reinforced. The cost was about $771 000. When

the contract was let in February, 1906, Mr. Christian was placed in charge of the construction, and remained in charge until the work was entirely completed in May, 1910. A length of several hundred feet of the tunnel sewer was constructed through what was locally known as the "Old Black Swamp" where the rock surface fell below sub-grade. It was necessary to use a shield and heavy timbering in driving this part of the tunnel, although the use of compressed air was not required. Mr. Christian gave his best thought to the design and construction of this work, at that time one of the largest concrete structures of the kind in the United States. While the plans were being prepared, he made a careful study of engineering literature relating to the subject and consulted freely with other engineers whose experience in such work would help him. He worked early and late on the job, and its success was due in large measure to his intelligent and faithful supervision of it and to his indefatigable energy. One of his associates on this work has stated that Mr. Christian was often to be found on duty during 18 of the 24 hours of the day and was subjected to great mental as well as physical strain, and that "this work will stand as a perpetual monument to George L. Christian as an engineer and citizen."

Subsequently, he had charge, as Assistant Engineer, of the construction of the large outlet sewer in Lacombe Avenue, emptying into the Bronx River. This was known as the Metcalf Avenue Sewer and cost about $575 000; it has been described in an article* written by Mr. Christian. During his connection with the Borough of the Bronx, he designed sewers aggregating in value more than $1 000 000, and supervised the construction of others of an aggregate value of $3 000 000.

On May 25th, 1915, he was transferred to the Bureau of Contract Supervision of the Board of Estimate and Apportionment of the City of New York, remaining there until the following December, when he became connected with the Bureau of Sewers, Borough of Manhattan. While with the latter Bureau he was in charge, among other contracts, of the Dyckman Street Screening Plant and of the reconstruction of the First Avenue sewer system in the Harlem District. The Thompson Street sanitary sewer system, with an automatic pumping plant at Canal Street, was begun, but not completed, under his supervision.

After twenty years of service with the City of New York, Mr. Christian was appointed Deputy City Engineer of Yonkers, N. Y., on August 28th, 1917. He remained in that position until his death, in general charge, under the City Engineer, of the design and construction of the engineering work, including waterworks, sewers, paving, regulating and grading, bridges, and docks for this city of more than 100 000 inhabitants. Of his record, Lawrence Griffith, Assoc. M. Am. Soc. C. E., City Engineer of Yonkers, writes: "Mr. Christian proved to me, in the official relationship that existed between us, that he was a man of strong character, a faithful employee, and a real friend."

His death came suddenly from heart failure on Sunday morning, April 25th, 1920, at his home in Yonkers. He had worked at his desk as usual the day before and, therefore, was active to the end.

Mr. Christian was active in the affairs of the Society and contributed to the discussion of papers which have appeared in the *Transactions*, on the following subjects: the Albany Filtration Plant; the consumption and waste of water; earth

* *Engineering News*, December 10th, 1914.

pressure and bracing; legitimate use of water; new water-works of Havana, Cuba; stadia topographic surveys; the water-works of Syracuse, N. Y.; the water-works of Los Angeles, Cal.; theory of concrete; and the water-works of Porterville, Cal. He also wrote a number of articles for the technical press, including one on the Tinton Avenue Sewer,* and the description of the Metcalf Avenue Sewer previously mentioned.

Mr. Christian was a Member of the Municipal Engineers of the City of New York, joining that Society of technical employees of the city on April 25th, 1906, and serving as a Director for two terms, from 1909 to 1911 and from 1913 to 1915. He was a Knight of the Golden Eagle and Past Master of Hopewell Lodge No. 596, F. and A. M., a member of the Royal Arcanum, and at the time of his death he was President of the Tremont (New York City) Building and Loan Association. He was a Warden and Financial Secretary of the Woodlawn Methodist Episcopal Church, in which he took a deep interest and whose pastor, the Rev. William R. Blackie, was his warm friend.

Mr. Christian was always interested in the affairs of his community. He had long been a member of the National Guard of the State, serving first as Private in the 4th Separate Company of Yonkers, and afterward in the 22d Regiment, joining as Private and being honorably discharged as Sergeant on October 30th, 1905. On May 5th, 1908, he was commissioned First Lieutenant, Coast Artillery Corps, N. G. N. Y., and was assigned as Ordnance Officer, 8th Coast Defense Command. He was honorably discharged on April 8th, 1910. He was also a member of Hope Hook and Ladder Company of the Yonkers Volunteer Fire Department.

Like many other good men, Mr. Christian felt no desire for the limelight of publicity. His work was done effectively in a quiet way, inspired by a deep sense of responsibility to his Profession and to his community. Modesty was an inborn trait and this was recognized by his associates, as well as his conscientious devotion to his ideals. In some lines of activity in the complicated life of to-day Mr. Christian's accomplishments would undoubtedly have brought much more prominence to him and public recognition of his work, but it is doubtful whether this would have given him the deep sense of satisfaction that came to him from the consciousness of work well done. Those who had the vision to see into the heart and mind of this modest man share with the writer a high regard for him as an upright public servant, a good citizen, and a loyal friend. It can be truly said that his death was a distinct loss to his Profession and to the community.

On April 19th, 1893, Mr. Christian was married to Miss Charlotte I. Prote, of Yonkers, N. Y., who, with three children, survives him. He is also survived by his mother and four sisters.

Mr. Christian was elected an Associate Member of the American Society of Civil Engineers on March 6th, 1895, and a Member on October 3d, 1905.

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* *Engineering Record*, November 7th, 1914.

THEODORE COOPER, M. Am. Soc. C. E.*

DIED AUGUST 24TH, 1919.

Theodore Cooper was born on January 13th, 1839, at Cooper's Plain, Steuben County, N. Y., where his father, John Cooper, Jr., born at Easton, Pa., was a practicing physician. His mother, Elizabeth M. Evans, was born at Wilkes-Barre, Pa. His parents were among the early settlers of Steuben County—moving there immediately after their marriage—and spent their lives developing an estate derived from the father's maternal grandfather, Arthur Erwin, and raising a family of nine children.

Theodore Cooper's American ancestry belonged solely to Pennsylvania, New Jersey, Delaware, and Maryland. His forefathers came to this country between 1630 and 1768 and were of English, Scotch, Irish, Welsh, Dutch, French, and German extraction; Quakers, Presbyterian, and Episcopalian in faith; and farmers, weavers, millers, mariners, and ministers in occupation.

Mr. Cooper was graduated as a Civil Engineer from the Rensselaer Polytechnic Institute in 1858, and was an Assistant Engineer on the Troy and Greenfield Railroad and Hoosac Tunnel in 1861.

At the outbreak of the Civil War, he entered the United States Navy in the fall of 1861, as Assistant Engineer, and was ordered to the gunboat *Chocura,* then building at Boston, Mass. In March, 1862, this ship joined the fleet at Hampton Roads, Va., and was present at the second appearance of the Confederate ironclad, *Merrimac;* took part in the siege of Yorktown, the battles of West Point and York River, and accompanied the Army of the Potomac up to the White House, where it remained as a guardship during the Chickahominy Campaign. The *Chocura* also took part in the blockade off Forts Caswell and Fisher from December, 1862, to August, 1863, as well as in the blockade along the Texas Coast from December, 1863, to March, 1865. In June, 1865, Mr. Cooper was detached from the *Chocura* and ordered to the Naval Academy, then at Newport, R. I., having been attached to the *Winnipeg* during the summer cruise as Instructor to the graduating class in practical steam engineering.

On the re-opening of the Academy at Annapolis, Md., in the fall of 1865, he was detailed as Instructor to the new Department of Steam Engineering and also as Secretary to the head of the Department to assist in the organization of the new course of studies. During his three years' duty at the Academy, Mr. Cooper also had charge of all the new construction, consisting of a sea-wall and a number of new buildings. In 1868, he was ordered to the *Nyack* in the South Pacific where he served for two years. He then returned to the United States, and, in September, 1870, he was again ordered to duty at the Naval Academy. After serving there a short time, he applied for a furlough and, finally, in July, 1872, resigned from the Navy as First Assistant Engineer.

In May, 1872, Mr. Cooper was appointed by the late Capt. James B. Eads, M. Am. Soc. C. E., as Inspector, at the Midvale Steel Works, for the steel being made for the St. Louis Bridge, then as Inspector of construction at the shops of the Keystone Bridge Company and, later, he was sent to St. Louis, Mo., in charge

* Memoir prepared by G. S. Greene, Jr., M. Am. Soc. C. E., and the late John Bogart, M. Am. Soc. C. E.

of the erection of the St. Louis Bridge. After the completion of the work, he was appointed as Engineer of the Bridge and Tunnel Company, succeeding Capt. Eads. Mr. Cooper's responsible and successful duties in connection with this important work are fully set forth in the "History of the St. Louis Bridge", by S. M. Woodward, M. Am. Soc. C. E.

The successful erection of the St. Louis Bridge by the cantilever method was the pioneer work of magnitude of this kind. The two half arches, extending from the river piers over a distance of 540 ft., exceeded any other similar construction until 1889, when the Forth Bridges were under erection.

In 1875-76, Mr. Cooper was Superintendent of the shops of the Delaware Bridge Company, at Phillipsburg, N. J., and, later, became Superintendent and Assistant General Manager of the Keystone Bridge Company, at Pittsburgh, Pa. He designed and built the Laredo Shops of the Mexican National Railroad; remodeled and rebuilt the furnace plant of the Lackawanna Coal and Iron Company, at Scranton, Pa.; and designed and built the Norton Cement Mills at Binnewater, N. Y.

Having established himself as a Consulting Engineer in New York City, in 1879, Mr. Cooper designed the Seekonk Bridge, at Providence, R. I.; the Sixth Street Bridge, at Pittsburgh; the Second Avenue Bridge over the Harlem River, in New York City; the Newburyport Bridge over the Merrimac River, and the Junction Bridges over the Allegheny River.

In 1894, he was appointed by the late President Cleveland as a member of a board of five engineers to determine the span of the Hudson River Bridge. Other consulting work included the New York Public Library and the Quebec Bridge, as well as work for the Suburban Rapid Transit Company, the New York Rapid Transit Commission, the Boston Rapid Transit Commission, and the Harlem River Commission. Mr. Cooper retired from active practice in 1907, having been, for more than a quarter of a century, a foremost authority on bridge design and construction.

Mr. Cooper was a member of the Military Order of the Loyal Legion, the New York Chamber of Commerce, the New York Historical Society, and the Century Association. He was also a Life Member of the Naval Institute and the Society of Fine Arts, and a Corresponding Member of the Institute of Architects.

He was a frequent contributor of technical papers at meetings of the Society, and, on two occasions, received the Norman Medal, his papers, "The Use of Steel for Bridges"*, contributed in 1879, and "American Railroad Bridges"†, in 1889, being so distinguished.

To quote from *Engineering News-Record*‡:

"An important contribution of Mr. Cooper to railroad bridge construction was his 'General Specifications for Iron Railroad Bridges and Viaducts', published in 1884, forming the first authoritative specifications on bridge construction that had been published and circulated. Those first specifications were revised from time to time and republished, along with specifications for highway bridges and for foundations and substructures of highway bridges and electric-railway bridges. The seventh edition of his Steel Railroad Bridge Specifications was issued in 1906.

* *Transactions*, Am. Soc. C. E., Vol. VIII (1879), p. 263.
† *Transactions*, Am. Soc. C. E., Vol. XXI (1889), p. 1.
‡ August 28th, 1919, p. 443.

"Even more than through his specifications, however, Mr. Cooper's name became universally familiar to bridge engineers through his system of locomotive and train loading for bridge design. Composed of a wheel system representing the heaviest locomotives of that time, followed by a uniform load whose amount in pounds per foot bore a simple relation to the driving-axle load, this system proved so convenient, and was so excellently adapted to modification for increasing weight of trains and engines by simple multiplication, that it quickly won a commanding position, and for many years now has been the almost universal standard for railway bridge design in America. Theodore Cooper also exerted a strong influence toward bringing about the adoption of wheel-load analysis for railway bridges in place of uniform-load or other methods, and the moment tables which he published made it possible to carry out the analysis rapidly and conveniently."

He died on August 24th, 1919, at his home in New York City, after a long illness. He had never married.

Mr. Cooper was elected a Member of the American Society of Civil Engineers on March 4th, 1874, and served as a Director in 1884 and 1885.

JOSEPH HOOKER CUNNINGHAM, M. Am. Soc. C. E.*

DIED DECEMBER 5TH, 1920.

Joseph Hooker Cunningham was born at Danville, Ill., on May 24th, 1863. He began his engineering career as an Assistant in the office of the City Engineer of Danville in 1883. Three years later he left this position to become Resident Engineer on the Mobile and Birmingham Railroad on the construction of that line between Mt. Vernon and Jackson, Ala.

In 1887, Mr. Cunningham went to the Pacific Coast as Assistant Engineer in the office of the City Engineer of Los Angeles, Cal., in charge of street and sewerage work. From that time until his death he was engaged in the active practice of his profession in the Western States.

In the early Nineties he located in Portland, Ore., as an Assistant Engineer on railway construction in Oregon and Washington and as an Assistant Engineer, U. S. Army, in charge of surveys and improvements on the Willamette River from Eugene to Portland.

From 1894 to 1899, Mr. Cunningham was employed on municipal work in the office of the City Engineer in Salt Lake City, Utah, and Los Angeles, Cal. Resuming his private practice in Portland in 1899, he designed and constructed a sewer for Oregon City and a water system for Pendleton, Ore.

From 1901 to 1904, he was employed as Principal Assistant Engineer for the Puget Sound Traction, Light and Power Company on investigations for hydro-electric developments, and on the construction of the plant at Electron, Wash., costing $3 000 000.

During the last sixteen years of his life, Mr. Cunningham maintained an office as a Consulting Engineer in Portland, his principal work being along hydraulic lines. He acted as Consulting Engineer to the City of Tacoma, Wash., on gravity water supply and hydro-electric plants.

He designed and built water and sewerage systems for Potlach, Idaho, and served as Consulting Engineer on various works for the Cities of Portland, La Grande, Pendleton, Salem, Oregon City, in Oregon, and Raymond, Wash.

Mr. Cunningham was a man of high character and scrupulous integrity, and by his death the Engineering Profession has sustained the loss of a member of wide experience and ability. He is survived by his wife and two sons.

Mr. Cunningham was elected an Associate Member of the American Society of Civil Engineers, on September 6th, 1899, and a Member on February 2d, 1904.

* Memoir prepared by the following Committee of the Portland, Ore., Section of the American Society of Civil Engineers: W. S. Turner, M. Am. Soc. C. E., and Ben. S. Morrow, Assoc. M. Am. Soc. C. E.

GEORGE ELVIN DATESMAN, M. Am. Soc. C. E.*

DIED OCTOBER 18TH, 1920.

George Elvin Datesman was born in West Milton, Union County, Pa., on January 20th, 1863. He was the son of Charles F. and Emma Good Datesman.

His early education was obtained in the public schools, including the High School, of Mahanoy City, Schuylkill County, Pa. He entered Lafayette College in September, 1879, and was graduated with honors in June, 1883, with the degree of C. E.

From the fall of 1883 until the spring of 1884, Mr. Datesman was employed in the architectural office of Mr. W. L. Plack, at Des Moines, Iowa, as a Student Architect. He resigned from this position to enter the employ of the Union Improvement Company of Jeddo, Pa., on engineering and surveying work, from which position he resigned on February 1st, 1885, to accept an appointment as Draftsman in the Bureau of Surveys of the City of Philadelphia. He held this position until March 1st, 1888, when he resigned to become associated with Mr. John H. Dye in the private practice of engineering and surveying in Philadelphia, Pa. On March 1st, 1891, Mr. Datesman gave up this private practice and returned to the Bureau of Surveys as a Draftsman. On March 23d, 1893, he was appointed to the position of Principal Assistant Engineer of the Bureau of Surveys, which position he held until January 3d, 1916, when he was appointed Director of the Department of Public Works of the City of Philadelphia. After the expiration of his term as Director, on January 5th, 1920, he became associated with Percival M. Sax, M. Am. Soc. C. E., in private engineering practice. In the following March, Mr. Datesman suffered a general physical collapse which terminated in his death on October 18th, 1920.

As Principal Assistant Engineer of the Bureau of Surveys, Mr. Datesman rendered valuable assistance in the design and supervision of many important public works for the advancement and improvement of the City of Philadelphia. He was particularly active in the studies for and the preparation of a comprehensive plan for the collection, purification, and disposal of the sewage of the entire city, as required by an Act of Assembly of the State of Pennsylvania. This work comprised the design, construction, and operation of a sewage treatment experiment station; a preliminary and final report for the collection and treatment of the sewage of Philadelphia; and the design, construction, and operation of the Pennypack Creek Sewage Pumping Station and Treatment Works. These works were the first step in the project for sewage treatment in Philadelphia and were constructed for the purpose of preventing the pollution of the main water supply of the city by the sewage discharged from several large public institutions and from the adjacent built-up territory known as Holmesburg Village. This plant has been in successful operation since 1912.

In the summer of 1913, in connection with the execution of a special mission for his Honor, the Mayor, that of installing, at the International Building Trades Exhibition at Leipzig, Germany, an exhibit comprising examples of comprehensive city planning of the City of Philadelphia, Mr. Datesman was commissioned

* Memoir prepared by Charles Frommer, M. Am. Soc. C. E.

to visit Europe and examine the systems of sewage treatment in representative cities of Germany, Austria, Holland, Belgium, France, and England. He visited forty-one cities, making a thorough investigation and comparison of the methods adopted by them for the treatment of sewage. The information obtained on this trip was made the subject of a report to the Department, part of which was incorporated in the final report of the Survey Bureau published in 1914, and resulted in valuable suggestions for the final plan for the treatment of the sewage of the entire city, of which a section of the first intercepting sewer is nearing completion and the first unit of treatment works is under construction.

Soon after his return from Europe, the Metropolitan Sewerage Commission of New York requested the city officials of Philadelphia to permit Mr. Datesman to examine and report on a proposed scheme for sewage disposal for the City of New York. This request was only one of many instances in which Mr. Datesman was recognized as an authority on sewage treatment. His report on the proposed New York project was a credit to himself and the city which he represented.

Included in his many other activities for the City of Philadelphia, as Principal Assistant Engineer, Mr. Datesman shared the responsibility with the Chief Engineer in the design and construction of nearly 700 miles of sewers; the widening and improvement of Delaware Avenue, between Vine and South Streets, including the construction of a concrete bulkhead and three large river piers; the dredging of the Delaware and Schuylkill Rivers; and the abolishment of the old Aramingo Canal and its improvement as a street from Norris Street to the Delaware River, requiring the construction of a concrete river bulkhead and large concrete twin sewers on piles.

He was also called into consultation on problems for the elimination of grade crossings; the design and construction of many bridges; the comprehensive plans for the revision and extension of the street system, including the Parkway and Boulevards and the many minor problems of routine and policy affecting the Bureau of Surveys.

As Director of the Department of Public Works, Mr. Datesman was a member of the Mayor's Cabinet and in direct charge of and responsible for the activities of the Bureau of Surveys, the Bureau of Water, the Bureau of Highways, the Bureau of Street Cleaning, the Bureau of City Property, the Bureau of Lighting, and the Bureau of Gas, for a period of four years. These bureaus comprise, as their titles indicate, the greater part of the engineering, operating, and maintenance forces of the City of Philadelphia, requiring exceptional ability as an engineer and executive to cope successfully with the multitudinous problems to be decided on daily. Mr. Datesman possessed these qualifications in a marked degree and performed the difficult duties of this high office conscientiously and competently.

He devoted the greater part of his life to the interests of the City of Philadelphia where he was known as a man of good character and high ideals, which, together with conscientious service and interest in his profession and public affairs, made him an efficient public official. He was of a modest nature, devoted to his home and family, and his amiable disposition endeared him to his associates and his many other friends to whom his death is an irreparable loss.

On November 19th, 1885, Mr. Datesman was married to Miss Minnie Gould Osterstock, of Easton, Pa., who, with his father and three daughters, Mrs. C. Elmer

Townsend and Mrs. L. O. Howell, 3d, of Philadelphia, and Mrs. Howard Dignan, of San Francisco, Cal., survives him.

He was a member of the Engineers Club of Philadelphia, the American Public Health Association, the Historical Society, and the Whitemarsh Country Club.

Mr. Datesman was elected a Member of the American Society of Civil Engineers on February 4th, 1903.

ADNA DOBSON, M. Am. Soc. C. E.*

DIED MAY 4TH, 1919.

Adna Dobson was born in Darlington, Wis., on June 7th, 1857, of English and Canadian parentage. In 1870, his parents moved to Lincoln, Nebr., where he received his High School education, having been a member of the first class which was graduated from the Lincoln High School.

In 1874, he entered the office of J. P. Walton, Civil Engineer, and at that time County Surveyor of Lancaster County, Nebraska, and took up the study of civil engineering, serving as Deputy County Surveyor and Assistant City Engineer of Lincoln for most of the period from 1876 to 1889. During this time the City of Lincoln grew from a village to a city of the first class, and its wide, beautiful streets bear evidence of Mr. Dobson's careful and conscientious work.

In April, 1891, he was appointed City Engineer of Lincoln, which position he held by re-election until April, 1896.

From 1896 to April, 1900, he was engaged in irrigation and engineering work, part of the time for the United States and State Governments, and part of the time in private practice. During this period, Mr. Dobson was called on by the citizens to act as their representative on the Councilmanic Board of the City of Lincoln. In April, 1900, he was again elected City Engineer, which position he held until July, 1901, when he resigned to become State Engineer of Nebraska, which position he held until April, 1909. In May, 1909, he was again elected City Engineer of Lincoln, and held this position by election and appointment until his death on May 4th, 1919.

During more than forty years of practice as a civil engineer, with only short periods of interruption, Mr. Dobson was a public servant, serving the County, City, State, and Federal Governments with sterling integrity and ability, particularly during the period of constructing the Commonwealth of Nebraska from what was the Great American Desert. The transition was slow and beset with obstacles; money was needed for irrigation works; riparian rights, as held by the old English common law, prevailed in the State, which virtually precluded private capital from safe investment. As Secretary of the Nebraska Engineering Society, Mr. Dobson was untiring in his efforts to pass the District Irrigation Law which, in reality, abrogated the riparian rights theory and established the "greatest good to the greatest number" theory, and resulted in a great impetus to the construction of irrigation work in the arid portion of the State. As State Engineer, he made many adjudications in contested claims for water, and his decisions were of such a character that, to the writer's knowledge, no Appellate Court has successfully reversed any of them. His death removes an engineer who, through his activities, had done much to develop the Arid West and bring it to a degree of prosperity far beyond the brightest expectations of his contemporaries.

During the sixteen years that Mr. Dobson served as City Engineer of Lincoln, he was a close student of municipal affairs and methods of construction, as is evidenced by the substantial improvements of that city.

* Memoir prepared by George L. Campen, M. Am. Soc. C. E.

Caution and conservatism were his predominating characteristics. His counsel and help to young engineers was well known and greatly appreciated by the students and Engineering Faculty of the University of Nebraska. Mr. Dobson was a member of Lincoln Lodge No. 19, A. F. and A. M.

He was married, in 1880, to Ellen E. North, who, together with two sons, F. S. Dobson and Arthur A. Dobson, survives him.

Mr. Dobson was elected a Member of the American Society of Civil Engineers on May 6th, 1908. He also served as President of the Nebraska Section of Members of the Society in 1919.

WILLIAM THOMAS DOUGAN, M. Am. Soc. C. E.*

DIED FEBRUARY 3D, 1919.

William Thomas Dougan was born at Mineville, N. Y., on November 12th, 1869. He attended the village school and, later, Sherman Academy at Moriah, N. Y., entering Union College in the fall of 1888, from which he was graduated with the degree of Civil Engineer in 1892.

Obtaining leave of absence from college in the spring of 1892, Mr. Dougan accepted a position as Transitman with W. Boardman Reed, M. Am. Soc. C. E., who was engaged in making surveys for the construction of an electric railway from Fonda through Johnstown to Gloversville, N. Y. This railway is now a part of the Electric Division of the Fonda, Johnstown, and Gloversville Railroad and was one of the first electric roads built on private right of way and designed for freight as well as passenger business. When construction work began, Mr. Dougan was appointed a Division Engineer, acting as Principal Assistant to his Chief, Mr. Reed.

In 1895, he served as Chief Engineer of Construction of the Herkimer, Ilion, and Frankfort Railway Company, now a part of the Utica and Mohawk Valley Railway System, and, on the completion of this work, he made surveys for an electric railway from Gloversville to Mountain Lake.

In 1896, Mr. Dougan came to New York City as Division Engineer for the Metropolitan Street Railway Company, which was then engaged in changing horse-car lines and cable lines to electric. He continued in this capacity until 1905, when he was again appointed Assistant to Mr. Reed—his old chief—who was then Engineer of Maintenance of Way and Buildings for the New York City Railway Company which operated all the surface lines in the Borough of Manhattan.

In 1906, he succeeded Mr. Reed in this position, in which he continued under the Receivers of the Company until October, 1915, when he became President of the T. H. Reynolds Construction Company. Mr. Dougan resigned this latter position in 1918, to engage in special engineering and contracting work, but ill health forced him to retire and he died on February 3d, 1919.

In 1893, Mr. Dougan was married to Clara Eisenbrey, of Gloversville, N. Y., who, with three children, Edward Hartley, William Rolland, and Marion Jane, now Mrs. Bleckwenn, survives him.

Previous to becoming Engineer of Maintenance of Way and Buildings for the New York City Railway Company, Mr. Dougan's work was that of a civil engineer working under the direction of others, but though his position as head of the Maintenance Department of the Street Railway Company called for the solution of many engineering problems, his work was largely that of an executive, having charge of not only men engaged on railway maintenance, but of craftsmen of several trades, including electricians, plumbers, carpenters, bricklayers, and painters.

* Memoir prepared by W. Boardman Reed, M. Am. Soc. C. E.

Always a hard worker and possessing engineering knowledge, a store of common sense, much tact in the handling of men, and being practical, rather than theoretical, Mr. Dougan was able to manage his department through times when funds for properly carrying on the work could not be had, in such a manner as to commend himself to his superior officers.

Mr. Dougan was elected a Member of the American Society of Civil Engineers on October 2d, 1907.

WILLIAM PIERSON FIELD, M. Am. Soc. C. E.*

DIED AUGUST 1ST, 1920.

William Pierson Field was born on August 7th, 1862, in Newark, N. J. He was the son of the late Moses and Anna J. (Pierson) Field, and came of a long line of distinguished ancestors, who were among the earliest settlers of Newark. He was a direct descendant of the Rev. Abraham Pierson, the first minister of the old First Church, and of his son, Abraham Pierson, the first President of Yale College.

After receiving his preparatory education, Mr. Field entered Princeton University, from which he was graduated in the class of 1883, with the degree of C. E. While at Princeton, he was a member of the Ivy Club.

Immediately after his graduation, in July, 1883, he joined the Engineering Corps of the Pennsylvania Railroad as Rodman in the Maintenance of Way Department, with headquarters at Jersey City, N. J. He was promoted at various times until April, 1886, when he was appointed Assistant Supervisor of Division A (Jersey City to Newark) of the New York Division. In this capacity he was in charge of double-track construction from Rahway to Perth Amboy, construction of masonry culverts, railroad stations, etc.

On July 1st, 1888, Mr. Field was granted a leave of absence by the Railroad Company, in order to take a course in engineering at the Royal Polytechnic School of Württemberg, in Stuttgart, Germany. On his return to the United States, he was made, on November 1st, 1889, Assistant Engineer to the Engineer of the New Jersey Division of the Pennsylvania Railroad, resigning on July 1st, 1890, to open an office in Newark, N. J., for the private practice of his profession.

In the course of his practice Mr. Field designed and supervised the construction of many and extensive works, which involved difficult problems in various lines of engineering. He was among the first to recognize the value of, and to use, reinforced concrete for building construction, and made a special study of that subject.

Among the many buildings and industrial plants which he had designed and constructed were: A coal terminal for the Tegen and Wiebke Company of Newark; the engineering design and supervision of the erection of State Armories at Jersey City, Paterson, Camden, Trenton, and Newark; the Lawyers' Building and the Progress Club Building, both of Newark; as well as buildings for Battin and Company, the First Presbyterian Church, and Trinity Church of the same city. Among his later works, was the design and construction of groups of buildings for Colgate and Company, the Essex County Park Commission, the Crocker-Wheeler Company, and the Oneida Community, Limited, of Oneida, N. Y. At the time of his death, he had not quite completed the construction of an industrial plant for Smith Brothers at Michigan City, Ind. He had also acted as Consulting Engineer for the Irvington Water Company of Irvington, N. J., and the Watchung Water Company of Dunellen, N. J., on service pipe lines, etc.; and was retained by the Broad Street Property Owners Association in Newark, to protect its interests, in the laying of wood block pavement on Broad Street.

*Memoir prepared by Karl G. Smith, Assoc. M. Am. Soc. C. E.

Mr. Field had devoted much of his time during his later years to his consulting practice.

He will be remembered by his friends, by his clients, and by those whose work he supervised, as a man of the most scrupulous integrity, a careful and painstaking engineer, and an inspiring personality. His sympathies were wide, his outlook broad, and his regard for the usefulness and reputation of his profession was always a guiding factor in his life. His genial and companionable disposition made for him a host of friends in all walks of life, and to them his death has been a keen loss. No work was too small to engage his best efforts in its plan and execution, and his personal pride in his work made it a pleasure individually to design and supervise all details. He enjoyed to an unusual degree the confidence of all with whom he came in contact, and was conscientious to a fault in the discharge of his commissions. His keen mind, his wide experience, unfailing kindness, and willingness to assume responsibilities, brought to him many positions of trust, entirely apart from his practice, involving a burden which could not have been borne by one of lesser stature or less unselfishness.

Mr. Field was for many years a devoted member of the First Presbyterian Church of Newark, and had served on its Board of Trustees for twenty-one years, and as President of the Board for seven years, until forced to resign on account of ill-health.

He had always kept up his interest in the affairs of Princeton University, and was the First President of the Princeton Engineering Association. As a member of the Board chosen to direct the policy of its Engineering Course, Mr. Field was frequently called on to address the students along technical lines.

He was married to Miss Josephine D. Smith in 1887, and lived from that time until his death at 976 Broad Street, Newark, N. J. He is survived by his wife and a son, William Thayer Field, Jun. Am. Soc. C. E.

He was a member of the University Club, the Essex County Country Club, the Baltusrol Golf Club, and the Society of Colonial Wars.

Mr. Field was elected a Junior of the American Society of Civil Engineers on December 7th, 1887, and a Member on March 6th, 1901.

THEODORE BOYDEN FORD, M. Am. Soc. C. E.*

DIED FEBRUARY 2D, 1920.

Theodore Boyden Ford, the son of Daniel Mill Ford and Harriet (Boyden) Ford, was born in Taunton, Mass., on September 4th, 1868. When he was one year old, his parents moved to Bridgeport, Conn., where young Ford received his education in the public schools.

Upon leaving High School, he entered the City Engineer's office and studied Civil Engineering under the late Horace G. Scofield, City Engineer. Later, Mr. Ford was made Assistant City Engineer.

In 1900, he became Mr. Scofield's partner, and they conducted a general engineering business under the firm name of Scofield and Ford. After Mr. Scofield's death, which occurred in April, 1917, the business was continued with the assistance of Mr. Aubrey D. Fuller, who now succeeds Mr. Ford as Owner and Manager.

Mr. Ford was Secretary and Treasurer of the Burns Company, a general contracting firm which was formed in 1901. He was also a Trustee of the City Savings Bank, a Director of the Bridgeport Storage Warehouse Company, and principal owner and Secretary of the Pequonnock Company.

Of the important work done under his charge may be mentioned the large reservoirs for the Bridgeport Hydraulic Company, the construction of the Shelton Street Railway, the Connecticut Railway and Lighting Company's line through the Housatonic Valley, the lines of the Westport and Saugatuck Street Railway Company, and the water supply system for Bethel, Conn.

In 1910, Mr. Ford was appointed to the Board of Railroad Commissioners of the State of Connecticut by Governor Simeon E. Baldwin, and, in 1911, automatically became a member of the Public Utilities Commission, which Commission he served until July, 1915.

Mr. Ford's death occurred on February 2d, 1920, following an illness of a few days. He is survived by his widow, Laura (Webb) Ford, to whom he was married on September 4th, 1909.

Mr. Ford was an engineer of ability. His clear-sighted judgment made him particularly valuable as a city and State official and contributed largely to the success of his private undertakings. Generosity and loyalty were his strongest characteristics. His frank expressions resulted in some adversaries, but his judgment was dependable and generally proved to be correct.

He was a member of the Connecticut Society of Civil Engineers and also of the Seaside Club.

Mr. Ford was elected a Member of the American Society of Civil Engineers on December 6th, 1905.

*Memoir prepared by C. C. Elwell, M. Am. Soc. C. E.

WILLIAM GRIFFING FORD, M. Am. Soc. C. E.*

DIED JUNE 23D, 1919.

William Griffing Ford was born on February 5th, 1862, at Memphis, Tenn., the son of William Griffing Ford and Mary (Sanders) Ford. His early education was received at the Emerson Preparatory School, in Washington, D. C., from which, in 1877, he entered the United States Naval Academy with the memorable Class of 1881.

On leaving Annapolis, he was assigned for his first cruise to the U. S. S. *Richmond* of the Asiatic Squadron. His careful and systematic records of that cruise were well known to his shipmates. Before returning to America, he received news of the reduction of the number of naval officers and the consequent discharge of all except a few of his class. After leaving the service, Mr. Ford was attached for some time to the United States Coast and Geodetic Survey, engaged in part in charting the currents in the Straits of Florida.

From 1886 to 1889, he was Chief Engineer of the New York Fish Commission. Here, among other services, his surveys of the oyster beds about Long Island were of great importance to the protection of that industry.

In 1890, Mr. Ford entered private engineering practice and, as a member of the firm of Ford and Beach, was engaged until 1901, except during the war with Spain, in the design and construction of docks, tramways, sewers, and roads. About 1895, he joined with others in forming the Second Naval Battalion of the State of New York, becoming a divisional officer and, finally, Lieutenant Commander of the Battalion, with headquarters in Brooklyn, N. Y.

At the outbreak of hostilities with Spain, in 1898, Mr. Ford was made commanding officer of the "mosquito fleet" patroling the harbor entrance to New York City and the mine fields laid there to meet the contingency of attack. In June, the most seaworthy craft of the fleet were selected for blockade duty in Cuban waters, and Captain Ford, with a crew drawn largely from his Battalion, was detailed to the gunboat, *Sylvia,* with the squadron off Havana. On August 12th, the *Sylvia* occupied the blockading station next westward of the flagship, *San Francisco,* and was under fire from the heavy shore batteries near the Morro. Later, on the same day, the *Sylvia* carried dispatches, under a flag of truce, close to the Morro; and it was Captain Ford, as Executive Officer, who boarded the *Pinzon* which came out to receive them. Previous announcement having been made by the Spanish that any vessel flying a flag of truce would be sunk, the hazard was a real one.

After the war his continued devotion to the upbuilding of a strong and lasting organization for the New York Naval Militia will always reflect great credit to his memory.

Returning to civil practice, Mr. Ford became active in harbor work in Brooklyn and New York City. His firm was selected by the Board of Estimate to make triangulation surveys for the East River bridges then in contemplation. The methods adopted by him to insure extreme accuracy, and their successful results, were highly regarded by his brother engineers.

* Memoir compiled by Frederick C. Noble, M. Am. Soc. C. E., from information furnished by H. F. Cuntz, Esq.

In 1907, the Hon. George B. McClellan, Mayor of New York City, appointed Mr. Ford as a member of the Jamaica Bay Improvement Commission. He visited Europe and made its principal harbors the subject of a thorough study, leading to an accumulation of the most complete data pertinent to the project of making the Port of New York the greatest in the world by the comprehensive improvement of that great harbor resource, Jamaica Bay. Although of a retiring disposition, constantly desiring to avoid personal praise, nevertheless he was a persistent working element of the Commission. He prepared a very elaborate minority report comprehending the harbor engineering information gathered on his European trips. This minority report was the one eventually adopted by the City and approved by the Federal authorities. His sacrifices to this most conscientious work involved more than any one, except his intimate friends, has ever known. Great expense in the development and publication of the report was borne by him far in excess of any consideration he received. To his efforts were largely due the introduction and passage of bills safeguarding to the City the land rights needed to make the development ultimately practical.

Mr. Ford was not only an able, conscientious engineer and a good citizen, but a man of culture and a product in all respects worthy of the best traditions of Annapolis. Although his exclusion from a naval career must have been a severe blow, it was significant of his nature that he was not embittered, and thereafter gave much of his time and energy to the improvement of the Naval Militia. His personal qualities were in every way admirable. Uniformly courteous, generous, and hospitable, he attracted and kept many friends and never missed an opportunity to serve them.

In 1887, he was married to Matilda, daughter of Archibald Young, of Brooklyn. Mrs. Ford was killed in an accident in 1906.

Mr. Ford was long a resident of Brooklyn, and took an active interest in its civic affairs. He was a member of the Municipal Club and was one of the organizers of the Brooklyn Engineers' Club, which elected him to its presidency in 1902. About 1890, he published a periodical in New Utrecht advocating civic reform.

Through the years following his wife's tragic death, filled as they were with business reverses, worries, and the burden of ill-health, Mr. Ford bore himself with never-failing cheerfulness and a soldierly determination, only now evident, to carry his own troubles without permitting his friends to view any but the happier aspects of his life. It was characteristic of him that, although the malady which caused his death was one of long standing, it was not until he was removed to a hospital that he was known to be in immediate danger. His death occurred on June 23d, 1919, at Roosevelt Hospital, New York City. His nearest surviving relatives are three sisters.

Mr. Ford was elected a Member of the American Society of Civil Engineers on October 5th, 1904.

ESTEVAN ANTONIO FUERTES, M. Am. Soc. C. E.*

DIED JANUARY 16TH, 1903.

Estevan Antonio Fuertes, the son of Estevan and Demetria (Charbonnier) Fuertes, was born on May 10th, 1838, in San Juan, Porto Rico, where his father was in the Government employ. The boy received an academic education, looking to a career in the Army or the Diplomatic Service, in Porto Rico and Spain, having been graduated from Conciliar College of San Ildefonso (Salamanca Jurisdiction) with the degrees of Ph. B. and Ph. D. As he grew older, his inclinations toward Engineering as a profession, however, brought him to the United States where he entered Rensselaer Polytechnic Institute from which he was graduated with the degree of C. E., in 1857.

After his graduation Mr. Fuertes returned to Porto Rico and joined the Royal Corps of Engineers. In 1862, he was made Director of Public Works of the Western District of the Island, in which capacity he was engaged on the construction of lighthouses, bridges, etc.

In 1863, however, he returned to the United States and was appointed Assistant Engineer for the Croton Aqueduct Board. Later, he was made Engineer of the Board and as such prepared reports, etc., on the Croton Aqueduct. After leaving the employ of the Croton Aqueduct Board, Mr. Fuertes was engaged for some years in the private practice of his profession as a Consulting Engineer in New York City.

In 1870-71, he served as Engineer in Chief of the United States Expeditions to Tehuantepec and Nicaragua, the object of which was an inquiry as to the feasibility of a ship canal across the Isthmus. On his return to New York City, he was again engaged as Consulting Engineer in connection with numerous extensive enginering enterprises in North and South America. One of his most important later engineering commissions was an elaborate report, in 1894, to the authorities of the Province of Santos, Brazil, on the sanitation of the City of Santos, including sewerage, pavements, quarantine, disinfecting stations, etc.

In 1873, Mr. Fuertes began his service as Dean and Director of the Department of Engineering at Cornell University, which position he held until his resignation on account of ill-health on November 7th, 1902.

When he came to Cornell, Professor Fuertes set to work to build up the Department of Engineering (as it was then) into what the late R. H. Thurston, M. Am. Soc. C. E., described as "a professional school that should stand high as a model in combining the science and the art of engineering in its curriculum and as affording the ambitious and competent novice entrance into his profession as well prepared as the learning and best practice of the time should permit." How well Professor Fuertes succeeded in attaining his ideal is shown by the standing of the College of Civil Engineering of Cornell University, into which the Department was developed in 1891 under his efficient management, among the Profession and the positions of responsibility held by its graduates all over the world.

* Memoir prepared from information on file at the Headquarters of the Society.

He was the first to apply laboratory work to the teaching of engineering and to make it an integral part of the prescribed course, and it was through his efforts that the foundation was laid for the present-day splendid laboratory facilities for its technical work, of which Cornell is justly proud. Professor Fuertes also gave freely of his engineering services for the benefit of the University, such as planning the water supply for the Campus, including the reservoir and supply mains, the designing of the Gothic Arch northwest of Sage College, the grading of the University Grounds, etc.

In addition to his work at Cornell, Professor Fuertes was engaged in much outside work contributing to the benefit of the Profession. Realizing their application to engineering problems, he, for sixteen years, directed and recorded meteorological conditions, in connection with his researches, contributing from his own resources the money for the apparatus, clerical work, etc. Even after the establishment of the New York State Weather Bureau, toward which he had done so much, he continued for ten years or more to direct and extend its work.

After his resignation as Dean and Director of the College of Civil Engineering, Professor Fuertes had been appointed Professor of Astronomy in charge of the A. C. Barnes Observatory and was preparing for his new work, when he contracted a severe cold and died, at his home in Ithaca, N. Y., on January 16th, 1903.

Professor Fuertes was a man of striking personality. Affable, warm-hearted and sympathetic in his personal relations, and ever the courteous gentleman, his friends were numbered by the hundred among faculty, alumni, students, and townsmen. His sprightly humor and broad interests made him a charming conversationalist. In his work he was full of enthusiasm and determination.

Professor Fuertes had traveled widely in Europe and North and South America and his professional activity and prominence were shown by his membership in the following societies: Corresponding Member of the Royal Economic Society (Spain); Life Member of the American Institute; Corresponding Member of the Humboldt Society and of the Society of Geography and Statistics, both of Mexico; Vice-President of the Economic Section of the International Congresses on Navigation, 1892; Corresponding Member of the Société des Ingenieurs Civils (France); and Member of Sigma Xi. He had contributed much to the technical and scientific literature of his time, for which and for his work for the Profession he was honored with numerous decorations, medals, diplomas, etc.

Professor Fuertes was elected a Member of the American Society of Civil Engineers on February 17th, 1869, and served as a Director in 1892.

WALTER HADEN GRAVES, M. Am. Soc. C. E.*

DIED SEPTEMBER 25TH, 1919.

Walter Haden Graves was born at Bloomington, Ill., on September 9th, 1851. His lineage in America is traced† from Thomas Graves of England, who, with his family, landed in this country before 1645, locating at Hartford, Conn. John Graves, second in descent from Thomas, was a Surveyor, and established the boundary line between Weathersfield and Mattabassett (now Middletown), Conn.

Linus Graves, father of Walter Haden Graves, was born at Brookfield, Vt., and when a lad accompanied his family to Bloomington, Ill. He was married to Virginia Haden, of Virginia, and became a prominent merchant of Bloomington. In later life, he was President and a shareholder in the Bloomington Cemetery Association. He was a staunch Republican of Civil War times, and held many friendly conferences with President Lincoln during and after the Lincoln-Douglas controversies. He was also a close acquaintance of United States Senator David Davis, of Bloomington.

Walter Haden Graves was the eldest son of the family. He interrupted his educational course at the Illinois Wesleyan University to take special training in civil engineering in the Jacques Cartier University of Montreal, Que., Canada. Returning to the Illinois Wesleyan University, later, he was chosen Valedictorian of his class, and, in June, 1874, received the degree of Bachelor of Arts. Immediately following his graduation, he became a member of the Powell Survey, during the famous explorations of the Colorado River and Canyon, and when this Bureau was enlarged into the United States Geological Survey, he was employed as Topographer.

In different ways, Mr. Graves assisted in collecting data for the Government report on "The Arid Lands of the United States", issued in 1877 and 1878. He was thus led to realize the importance of irrigation in the West, and decided to make this his special study. In 1879, therefore, he resigned his position with the U. S. Geological Survey to locate in Denver, Colo., where he placed on his office door the sign "Irrigation Engineer". This is believed to be the first sign of this kind displayed in the United States. In the pioneer irrigation field, he was active in working out and constructing systems which set the pace and established precedents which later were accepted as standards among scientific irrigators.

In 1879, in association with *ex*-Governor Route of Colorado and Senator John S. Stanger, he planned and constructed the Empire Canal, the first of the great canal systems of the San Louis Valley in Southern Colorado.

About 1885, Mr. Graves was appointed Chief Engineer and Superintendent of the Colorado properties of the Travellers Insurance Company. In that capacity, he designed and constructed the Citizens and Rio Grande Canal Systems which supplied more than 500 000 acres with water, and cost approximately $2 500 000; also the canals about Grand Junction, which have made the Grand River Valley famous, as well as the Uncomphagre Canal in Western Colorado, which was after-

* Memoir prepared by Orrin E. Stanley, M. Am. Soc. C. E.

† "Graves' Genealogy", Vol. I.

ward purchased by the Government in connection with the "Gunnison Tunnel Project."

According to a treaty with the Crow Indians in Montana, the United States took over a part of their Reservation, agreeing in return to supervise the expenditure of $200 000 of the purchase price in irrigation projects on the remainder of their lands. This was during President Harrison's Administration and his Secretary of the Interior, the Hon. John W. Noble, appreciating the importance of the work, addressed some of the Governors of the Western States as to the proper man to have charge of the undertaking. A consensus of opinion recommended Mr. Graves, and he was invited to Washington where, in May, 1891, he furnished bonds and filed an oath of office as "U. S. Superintendent of Irrigation and Special Disbursing Officer."

Under this commission, Mr. Graves reported to Secretaries of the Interior John W. Noble, Hoke Smith, David Francis, and Cornelius Bliss, in the Harrison and Cleveland Administrations, while he was constructing the gigantic headgate works and canal of the Big Horn River, the Little Big Horn, and Pryor Creek in Montana.

The treaty required the employment of Indians at regulation rates, and their education for this work was added to Mr. Graves' engineering problems. He was assisted by an engineering party and a clerk.

In the midst of this project, Mr. Graves was again called to Washington to accept a commission from the late President McKinley described as an "Original Vacancy" carrying the official title of "U. S. Irrigation Engineer and Inspector". While in this service he constructed irrigation works of various kinds, including storage reservoirs, pumping plants on many of the Indian Reservations throughout the country, and also water supplies and sewer systems for the Indian schools. Voluminous reports were made by Mr. Graves to the Hon. Ethan Allen Hitchcock, and the substance of many of the ideas and suggestions in these reports was helpful in the formation of the U. S. Reclamation Department, organized a few years later. He resigned this position in May, 1902.

After a few years spent on his fruit farm in Idaho, Mr. Graves again entered his chosen field, and, in 1907, became President of the Northwestern Engineering Corporation, with offices in Spokane and Seattle, Wash., and Portland, Ore. While at the head of this company, he designed and built a unique system for watering the lands near Pasco, Wash., which were at such an elevation above the river that it had been thought impracticable to irrigate them. This was known as the Pasco Irrigation Project and was completed in 1911. Electrically operated pumps were installed for lifting the water to an elevation of 122 ft., to a reservoir from which it was carried to the lands to be irrigated through wood stave pipes.

The attention of Mr. Graves was continually called to the great areas of idle cut-over land left by the lumber industries, and he decided that this was as legitimate a field for reclamation as irrigation lands, and called for a high class of engineering skill. He located at Portland, Ore., and at the expense of a great deal of money and many years of work, perfected a practical method of clearing land by electricity.

Then came the World War, upturning all normal activities. After the United States became actively engaged in the conflict, Mr. Graves served as Engineer and Inspector in shipyards at North Bend, Ore., and at San Francisco, Cal.

In August, 1918, he was appointed Irrigation Engineer in the U. S. Reclamation Service, and was given charge of the "study of cut-over lands in the Northwest", including Montana, Idaho, Washington, Oregon, and California. This came under the Congressional appropriation bill for drainage and reclamation of cut-over lands and meant a cursory statistical survey of these States as to the area of land susceptible of reclamation, their possible crop results, methods for clearing, etc., in view of the anticipated demand for farm lands after the war. He handed in a final report of this work early in January, 1919.

During this last strenuous activity, Mr. Graves was attacked by the prevailing influenza. A weakened heart was the result, although its seriousness was unsuspected until the very last. His death occurred on September 25th, 1919, at Salinas, Cal., where he had gone the previous month to visit his daughter. Acceding to his repeated request for cremation, the ceremony was performed at Mt. Olivet Cemetery, San Francisco, Cal., on September 28th, 1919.

He is survived by his wife, Katherine R. Graves, daughter of the Rev. William Plummer Graves, of Pasadena, Cal., his daughter, Mrs. John G. Hunter, two brothers, Arthur Graves, of Bloomington, Ill., and Linus Graves, of Los Angeles, Cal., also two grandsons, Walter William Young and Richard Clyde Young, and two granddaughters, Theodosia Young and Helen Unity Hunter.

While living in Portland, Ore., Mr. Graves was active in the organization of the Oregon Society of Engineers, and was elected in 1913 to be its second President, serving as such for two years. In early manhood, he became affiliated with the Masonic Order and held a life membership in the Scottish Rite Lodge, in Washington, D. C.

Mr. Graves was elected a Member of the American Society of Civil Engineers on February 2d, 1909.

CHARLES EZRA GREENE, M. Am. Soc. C. E.*

DIED OCTOBER 16TH, 1903.

Charles Ezra Greene, the son of James Diman and Sarah A. (Durell) Green, and a direct descendant of James Greene, who resided in Charlestown, Mass., in 1634, was born in Cambridge, Mass., on February 12th, 1842. He attended the Cambridge High School and Phillips Exeter Academy and was graduated from Harvard College with the degree of A. B. in 1862.

His first work was in a shop engaged in the manufacture of breech loading rifles, but early in 1864 he became a clerk in the Quartermaster Department of the Army and, in 1865, was commissioned as First Lieutenant in the Seventh Regiment, U. S. Colored Troops, in which he served as Regimental Quartermaster for a year and a half in Virginia and Texas.

Upon leaving the Army, Mr. Greene decided to adopt the Profession of Civil Engineering, and accordingly entered the Massachusetts Institute of Technology, from which he was graduated in 1868 with its first class. After graduation, he was Assistant Engineer on the location and construction of the Bangor and Piscataquis Railroad in Maine, and upon its completion in 1870 he was employed under General Thom on United States river and harbor improvements in that State and in New Hampshire, which work he left to become City Engineer of Bangor, Me. In 1872, he was appointed to the Chair of Civil Engineering in the University of Michigan, and when a separate Department of Engineering was established, in 1895, he was made its Dean. He held both positions until his death, having served as Professor of Civil Engineering for thirty-one years.

Although teaching was Professor Greene's main vocation after 1872, he also did a considerable amount of designing and consulting work. He was Chief Engineer of the Toledo, Ann Arbor and Northern Railroad from 1879 to 1881, Consulting Engineer on the Wheeling and Lake Erie Railroad Bridge and on the Cherry Street Draw-Bridge, both at Toledo, Ohio, Designer of the Ann Arbor waterworks system in 1885, and of the Ann Arbor sewerage system in 1890.

Professor Greene was Associate Editor of *Engineering News* in 1876-77, when that periodical was in its infancy, and during that time wrote many of its editorials. He also wrote a series of papers on the graphical analysis of roof trusses which appeared therein. He was greatly interested in graphical methods for the analysis of structures, and did much to popularize their use in this country. In 1875 he published "Graphical Method for the Analysis of Bridge Trusses," in which he applied to continuous girders and draw-spans the method of "area-moments" which he had discovered and taught to his classes the preceding year. From 1876 to 1879, appeared his "Trusses and Arches" in three parts: Part I, Roof Trusses; Part II, Bridge Trusses; and Part III, Arches. Part II is a revision and extension of his earlier work. In 1897 he published "Structural Mechanics", a treatise on the strength of materials.

Professor Greene had the teacher's gift of clear and full presentation of a subject, and he wrote readily with little revision. Personally, he was a man of quiet and simple tastes and a conservative by nature. He was interested in the

* Memoir prepared by A. E. Greene, Esq.

progress of his adopted city, Ann Arbor, was a Director in a number of local business enterprises, and was Vice-President of the Farmers and Mechanics Bank of that place. For many years he was a Trustee of the Unitarian Church.

In 1872, he was married to Miss Florence Emerson, of Bangor, Me., who, with two children, Albert E. and Florence W., survives him.

Professor Greene was elected a Member of the American Society of Civil Engineers on January 4th, 1882.

FREDERICK THOMAS HATCH, M. Am. Soc. C. E.*

DIED MARCH 9TH, 1920.

Frederick Thomas Hatch, the eldest son of Nathaniel and Catherine Harbach Hatch, was born in Haverhill, Mass., on November 21st, 1855. His ancestors on both sides were among the earliest settlers of the Massachusetts Bay Colony. His early education was obtained in the school in Bradford (now Haverhill), Mass., and at Phillips Academy, Andover, Mass., from which he was graduated first in his class in 1874.

From 1874 until 1880, Mr. Hatch was with the Engineering Corps of the Boston, Hoosac Tunnel and Western Railroad, in connection with the construction of the line from Greenfield, Mass., to the Hoosac Tunnel, and of the predecessor of the present Missouri Pacific Railroad, having been at Pueblo, Colo., at the time of the armed fight between the Missouri Pacific and the Denver and Rio Grande Railroad Companies for an entrance into Pueblo.

From 1880 until the time of his death, Mr. Hatch was continuously with the Pennsylvania Lines West of Pittsburgh and its controlled line, the Vandalia Railroad, as follows: From 1880 to 1888, with the Maintenance of Way Corps of the Pittsburgh, Cincinnati and St. Louis Railway; 1888 to 1890, as Engineer of Maintenance of Way, at Indianapolis, Ind.; 1890 to April, 1894, as Engineer of Maintenance of Way, at Pittsburgh, Pa.; April, 1894, to March, 1903, as Superintendent of the Michigan Division of the Vandalia Railroad, at Logansport and Terre Haute, Ind.; 1896 to 1902, also as Chief Engineer of the Vandalia Railroad; December, 1901, to March, 1903, also as Superintendent of the Logansport and Toledo Division of the Vandalia Railroad; and from March, 1903, to January 1st, 1905, as Superintendent of the Peoria Division of the Vandalia Railroad, the position of Chief Engineer having been abolished.

He was again appointed Chief Engineer of the Vandalia Lines on January 1st, 1905, with headquarters in St. Louis, Mo., which position he held until the actual absorption of the Vandalia Railroad by the Pennsylvania Lines, when he became Chief Engineer of Maintenance of Way of the St. Louis System. It was during this time that a comprehensive programme for the improvement of the road was laid out, not only on the broad lines of a general plan, but in detail, in which Mr. Hatch's alert mind and practical judgment were of great value. These improvements included track elevation at Indianapolis, the construction of second track from Indianapolis to St. Louis, as well as increased yard and engine terminals and grade reductions. He was also actively interested in the proposed entrance of the Pennsylvania Lines into the Detroit territory.

Mr. Hatch's last contribution of engineering importance was the building of the Indianapolis and Frankfort Railroad, 41 miles long, now a part of the Louisville-Indianapolis-Chicago Line, of the Pennsylvania System, which he aimed to make first-class in every particular, anticipating the standards as well as the needs of the future. The maximum northward grade in the direction of the anticipated heavier traffic was fixed at 0.3% and the maximum southward

* Memoir prepared by George W. Kittredge, M. Am. Soc. C. E.

grade at 0.5 per cent. The grades are separated at all railroad and city street crossings and at nearly all country roads. The bridges over other railroads are steel structures, spanning their entire right of way in every case except one. The bridges over roads and streams are, with few exceptions, reinforced concrete structures, the exceptions being over drainage ditches where steel spans, which can be lifted from the supporting masonry to permit the passage of dredges, are used.

During the period of Federal control, Mr. Hatch was Consulting Engineer of the St. Louis System. The return of the railroad to private operation on March 1st, 1920, together with the re-organization of the Pennsylvania Railroad and Pennsylvania Lines, required a great deal of preparatory work at St. Louis, and a large part of this burden fell on his shoulders. Taken in conjunction with the effects of a previous illness, the strain was too great for him, and he died very suddenly at his home early in the evening of March 9th, 1920.

He was married to Alice Gertrude Hill, at Steubenville, Ohio, on August 13th, 1884, and is survived by a daughter and two sons, both of the latter having served their country in the World War and both having attained the rank of Major. His first wife died in 1905, and he was married to Mrs. Nola Pearson Underwood, at Mexico, Mo., on April 9th, 1916, who also survives him.

Mr. Hatch was a thin, spare man of marked personality and of untiring energy. He was honest, straightforward, and decided in his actions, and held the respect and esteem of those above and below him in authority; all who came in contact with him knew his attitude on subjects of moment.

The following tribute by J. J. Turner, Vice-President of the Pennsylvania Lines, is quoted as coming from an official who knew Mr. Hatch and had the opportunity of seeing him in daily action for many years and who is, therefore, qualified to judge the man:

"Mr. Hatch was a man who never strove for applause and when he did big things, such as the reconstruction of the bridge over the Tuscarawas River, near Newcomerstown, in 1893, with the water at flood stage during nearly the whole period, he was content to work 24 hours a day and to take his commendation out of his own feelings at having adroitly won the contest over the elements.

"I have often had described to me his actions during the strike in 1894, when he was Superintendent of the Michigan Division of the Vandalia Railroad. A memory that lingers in the minds of the people who were associated with him there, is his figure standing against the corner of the station at Terre Haute, so that he could look both ways, and of his great patience and kindly treatment of the misguided mankind. In 1896, at the time of the great cyclone at East St. Louis, he was a prominent figure in the work of reconstruction, and when he came away made no further comment upon it other than occasionally to relate amusing incidents that happened during the work. He went through the floods of 1913 with the same methodical pursuit of the desired end and with apparently no thought whatever that he occupied a position in relation thereto that was greater than the humblest man on the job, and I loved him."

He was a member of the American Railway Engineering Association, the American Railway Guild, and several local societies in St. Louis, and was a Thirty-second Degree Mason and a Shriner.

Mr. Hatch was elected a Member of the American Society of Civil Engineers on December 4th, 1889.

ALBERT JAMES HIMES, M. Am. Soc. C. E.*

DIED NOVEMBER 4TH, 1919.

Albert James Himes was born at Oswego, N. Y., on November 14th, 1864. After receiving the usual public school training in Oswego, where he was graduated from the High School in 1883, he entered Cornell University, having been awarded a scholarship in a competitive examination. At Cornell, he pursued the course in Civil Engineering, and was graduated with honor in 1887.

Immediately upon his graduation, Mr. Himes secured a position as Assistant Engineer with the Burlington and Missouri River Railroad in Nebraska, and was employed for several years on the construction of important bridges over the Missouri River at Nebraska City and Rulo. Following this service, in 1889, he was engaged as Assistant Engineer with the Fall Brook Coal Company Railway, and spent the following year (1890) on reconstruction work. After a short engagement on surveys for an extension of the Beech Creek Railway, he accepted, in 1891, the position of City Engineer of Corning, N. Y. Mr. Himes filled the position at Corning for two years, carrying forward the usual engineering work pertaining to such an office, and, in addition, by special arrangement, engaged in private practice—conducting surveys for a logging railroad, as well as for an 8-mile extension of the Fall Brook Railroad, and also building a dike in the Chemung River. As City Engineer, he introduced for the first time at Corning the practice of laying water and sewer mains before paving, and he also laid the first brick pavement there.

During the three years following July, 1884, he occupied the responsible position of Resident Engineer on the Eastern Division of the New York State Canals, the duties of which involved a variety of engineering work, including full charge of all field work and the preparation and approval of estimates for payment, as well as estimates for new appropriations. Owing to grave differences of opinion with his superiors in rank, growing out of changes in classification of material, ordered against his protest, and feeling that political pressure was being used to favor certain contractors, he resigned in 1897. In this connection, the late Alfred Noble, Past-President, Am. Soc. C. E., paid Mr. Himes a high tribute for his conscientious adherence to his standard of probity.

On leaving the State Canal service Mr. Himes was offered and accepted the position of Assistant Engineer on Deep Waterways Surveys in New York State. This project involved a thorough study of the route for a canal from Oswego to Herkimer and included a triangulation of Oneida Lake. The work being concluded in 1899, the following months up to January, 1901, were spent by him on various investigations including the Michigan Railroad appraisal under M. E. Cooley, M. Am. Soc. C. E.

Mr. Himes entered the service of the New York, Chicago and St. Louis Railroad in January, 1901, as Bridge Engineer, and in this capacity designed all the steel structures then being substituted for the obsolete types. This work included the design of masonry substructures and personal attention to shop, mill, and field inspection.

* Memoir prepared by the following Committee: Hunter McDonald, Past-President, Am. Soc. C. E., A. W. Johnston, and George H. Tinker, Members, Am. Soc. C. E., and Huntington Smith, Assoc. M. Am. Soc. C. E.

In January, 1907, he became Assistant Chief Engineer in charge of second track construction and yard enlargement, and was especially charged with the provision of plans for the elimination of grade crossings, in addition to bridge design.

On February 3d, 1909, Mr. Himes was designated to assume charge of all engineering problems involved in carrying into effect the contracts with the municipalities of Cleveland and East Cleveland, Ohio, for the separation of street grades, which, for several miles, included four-track construction to accommodate the Cleveland Short Line Railroad. Acting for all the interested parties, the plans were executed under an organization created by himself, all the work being done by direct labor.

This work occupied the greater part of four years, and was concluded to the entire satisfaction of the various interests, municipalities and railroads, and as completed reflected great credit on Mr. Himes and his staff. His careful preparation and methodical exactness in plans and systems of records enabled the interested parties to obtain clear and concise data and information of special value at any stage in the progress of the work.

In 1915, Mr. Himes was designated as Valuation Engineer of the New York, Chicago and St. Louis Railroad Company, in connection with his other duties. He built up the organization necessary to accomplish properly the purpose of the Valuation Act, under the rules and instructions of the Federal Valuation Division. In this relation the same thoroughness and painstaking care was exhibited as in all his other undertakings. He continued as Valuation Engineer until failing health compelled cessation from all duties. He died at Cleveland, Ohio, on November 4th, 1919.

Mr. Himes maintained a lively interest in his Alma Mater keeping steadily in touch with its affairs and aiding by membership and leadership in its Alumni organizations the maintenance of valuable relations between the college and its graduates.

He took great pride in his profession and sought by personal interest and service in the several technical societies covering the field of his activity, to advance their dignity and influence in the public mind.

Mr. Himes treated his membership in this Society as a serious obligation and endeavored to live up to its requirements. He contributed a paper* to the *Transactions* of the Society, and also discussions of papers presented by other members, and all matter presented by him gave evidence of careful preparation, exactness of statement being a cardinal point. He deprecated the use of inexact language and always sought by example and counsel to teach young men coming under his authority the habit of exactness in language, as well as precision in work. The assembling of facts and not guesses was always with him a preliminary step to the proper analysis of problems.

Mr. Himes was also an active member of the American Railway Engineering Association. Owing to his special fitness for such service, he was elected a member of its Committee on Iron and Steel Structures in 1906, and served continuously as such until 1917, having been Chairman of the Committee from 1912 until 1917.

* "The Position of the Constructing Engineer, and His Duties in Relation to Inspection and the Enforcement of Contracts," *Transactions*, Am. Soc. C. E., Vol. LVI (1906), p. 104.

During his membership of this Committee there were perfected improved general specifications for steel railway bridges, and collateral subjects of the highest importance were acted on; in all of which he had a leading part, and the reports of this Committee from year to year bear the impress of his systematic thinking. As Chairman of the Committee his ability in presenting and sustaining its conclusions in open convention, his full command of the subject under discussion, aided by logical statement carefully phrased, brought his pre-eminence into full recognition. He also served as a member of the Committee on Rules and Organization of the Association from 1916 to the time of his death.

Mr. Himes was also an active member of the Cleveland Engineering Society, which he joined in 1908, having served as its President in 1912-1913. He took a deep interest in its affairs and gave much time and effort to promote its growth and increase its prestige.

Due to his endeavor; there was started during his Presidency the publicity work of that Society, which has been so highly commended by other technical societies all over the United States.

He believed firmly in the mission of the engineer as a leader in the community, and, in the study of this phase of the engineer's duty and privilege, he became convinced that the engineer's usefulness to the community could only be increased to the degree that the public rose to an appreciation of his qualifications.

In furtherance of this aim to acquaint and bring home to the public the value of the engineer's services to the community, Mr. Himes promoted a special publication of the Cleveland Engineering Society, entitled the *Conquerors*. It was his belief that a high-grade magazine describing engineering achievement in language familiar to the layman, with a minimum of text and a maximum of illustration, would be widely read and of great mutual benefit in bringing a knowledge of the work of the engineer to the non-technical public. The reception accorded the *Conquerors* proved this belief to be well founded.

As a member of the several Engineering Societies referred to, Mr. Himes evinced a serious interest in their general purposes, which was reflected by his contributions in the form of papers, the more noteworthy of which were published in the *Bulletins* of the American Railway Engineering Association. Those dealing with the elimination of grade crossings in Cleveland and on the science of organization are especially interesting. They reflect the serious application and the systematic thinking which characterized all his professional work.

His other contributions were in the form of discussions—on the stresses in track fastenings, classification of existing bridges, and on rail end connections for draw-bridges.

Mr. Himes was married on February 28th, 1889, to Miss Grace Hyatt, who, with two sons and one daughter, survives him.

He was a member of the Presbyterian Church, at Corning, N. Y. In all his domestic and personal relations and in his official intercourse he left a lasting impression of an entirely sincere and conscientious man. He adhered steadfastly to whatever course he marked out in any line of activity, and with rare exceptions reached the goal. He was always "loyal to the job".

Mr. Himes was elected an Associate Member of the American Society of Civil Engineers on November 6th, 1895, and a Member on June 7th, 1899.

EDWARD HENRY HOLDEN, M. Am. Soc. C. E.*

DIED AUGUST 7TH, 1920.

Edward Henry Holden, the son of the late Rev. Dr. Robert Holden, Rector of Trinity School, was born at Sing Sing (now Ossining), N. Y., on November 18th, 1856. He was graduated from Trinity School and Columbia University, and received the degree of Civil Engineer from the School of Mines of the latter institution in 1878.

In 1878 and 1879, Mr. Holden was employed as Heliotroper on the triangulation of the State of New York, following which he was engaged for several years in the private practice of engineering and surveying in the upper part of New York City.

From 1885 to 1893, Mr. Holden was associated with Robert L. Waters, City Surveyor, as Engineer in charge of surveys of new parks in New York City, namely, Van Cortlandt, Bronx, Crotona, Claremont, St. Mary's, etc.

In 1893, he was appointed Assistant Engineer in the office of the Commissioner of Street Improvements, which office had charge of the engineering work of what is now the Borough of the Bronx, in charge of a field party making surveys for monumenting, tax maps, etc. Mr. Holden retained this position until 1899, when he was made Engineer in charge of surveys and monumenting in Greater New York.

In 1902, he was appointed as Engineer in connection with the same work in the office of the President of the Borough of the Bronx, which position he held at the time of his death. In this capacity he had had charge of the monumenting of the entire city for four years and afterward of the triangulation of the Borough of the Bronx. He had also acted as Consulting Engineer for the City of Yonkers, N. Y., in the matter of the triangulation of that city.

His death, which was due to heart trouble, occurred at his home on the Boston Road, New York City, on August 7th, 1920. He is survived by his wife and one daughter.

Mr. Holden had devoted much of his time to the perfection of a lineclinometer, an instrument which he had devised for use in measuring long triangulation base lines. This instrument serves as a measuring station, and by means of a telescope the difference in elevation between stations is determined immediately. He had also devised and patented a tape stretcher for use in measuring triangulation base lines with long tapes. By the use of this stretcher, tension can be applied to any given amount and held without variation.

Mr. Holden was a member of the Municipal Engineers of the City of New York. He was a member and, for many years, Senior Warden of St. Mary's Protestant Episcopal Church.

Mr. Holden was elected a Member of the American Society of Civil Engineers on May 6th, 1908.

* Memoir compiled from information supplied by John P. Binzen, Esq., and on file at the Headquarters of the Society.

ROBERT PARSONS HOWELL, M. Am. Soc. C. E.*

DIED SEPTEMBER 29TH, 1920.

Robert Parsons Howell, the only son of Mr. and Mrs. William C. Howell, was born in Blairstown, N. J., on September 28th, 1873. He entered Blair Academy in 1889, and completed his college preparatory course there in June, 1892. In the fall of that year he entered Lehigh University from which he was graduated in 1896, with the degree of C. E.

In July, 1896, Mr. Howell entered the service of the United States Geological Survey as Assistant Transitman until November, when he was appointed U. S. Surveyor in charge of a party on the subdivision of Indian Territory, remaining on that work until June, 1898.

From September, 1898, to January, 1899, he was engaged as Assistant Engineer on Construction with the Atlas Portland Cement Company, at Northampton, Pa.

In February, 1899, Mr. Howell entered the Hydrographic Department of the U. S. Navy, and until October, 1900, served as Topographer and Cartographic Draftsman on a survey of the rivers and harbors of Cuba. Up to that time, the maps and charts of Cuban waters were very imperfect, the shipping having been piloted to berths by natives familiar with the waters. Under Mr. Howell's direction, however, not only were surface surveys and maps made, but the bottoms of the harbors were charted, and rocks, sand-bars, and other obstacles to safe navigation were marked.

From January, 1901, to July, 1902, he was engaged on the construction of a railroad from Havana to Santiago, for the Cuba Railroad Company, first as Topographer on Location and afterward as Assistant Engineer in charge of a 30-mile residence.

Mr. Howell then returned to the United States and located in Phillipsburg, N. J., where from October, 1902, to October, 1903, he was engaged in private practice as a Consulting Engineer. It was during this time that he designed and built the Blair Dam, forming Blair Lake.

In October, 1903, he was appointed Town Engineer of Phillipsburg, in which position he had charge of the design and construction of a sewerage system for the town, all street improvements, mapping, etc. He also served as City Commissioner of Phillipsburg, and as Borough Engineer of Washington and Alpha, N. J., and continued his private practice as a Consulting Engineer.

When the United States entered the World War, Mr. Howell offered his services to the Government and on his previous record was commissioned a Captain in the Construction Division of the Quartermaster Corps. In this connection he served as Utilities Officer at Camp Dix, Camp Gordon, and the Port of Embarkation at Newport News, Va., in charge of water and sewerage projects.

After his discharge from the Army in August, 1919, Captain Howell spent the autumn and winter in Washington, D. C., and Florida in a much needed rest, and in May, 1920, returned to Blairstown, N. J., to complete the term as a teacher at Blair Academy. At the end of the school year, he was appointed as Assistant to the Headmaster of the Academy, Dr. John C. Sharpe. In this capacity one of

* Memoir prepared from information on file at the Headquarters of the Society.

his first duties was to have been the development of the grounds of the Academy, by road construction, landscape gardening, etc. This position appealed to him greatly because of its engineering possibilities, and he spent the summer in New Hampshire preparing for the work with characteristic pleasure and energy. Ill-health, however, compelled him to relinquish his plans and after a short illness he died at the Garfield Memorial Hospital in Washington, D. C., on September 29th, 1920.

Captain Howell was married in 1902 to Miss Bertha E. Wenner, of Allentown, Pa., who, with one daughter, Eloise Saeger, survives him. He is also survived by his father and mother and a sister, Mrs. Frank G. Wilkins, of Washington, D. C.

In writing of him, Dr. Sharpe says:

"A pen sketch is a feeble presentation of the character and ability of our friend. He had that rare and indefinable quality that made whatever his hand planted grow and blossom and bear its appropriate fruit. I suppose it was his love for plants, as God's creatures, that made his touch so gentle and skillful. I suppose also that it was his love for God's children that made his touch so gentle and helpful to every one that knew him. To him no one was young or old, rich or poor, each one was more than his fellow, he was his friend. What hosts of friends there were, when all who knew him were friends for him to know, to love, to help, in that consecrated life that was so wholly unconscious of the help he was giving from hour to hour, on every hand. His closest friends never knew him to speak ill of any one. He could not for he loved his fellowmen.

"He was big and strong in his manly six feet, and learned and skillful in his profession. We are proud of him and honor him in the great work he has accomplished. His work will live after him, but our hearts turn not to the Captain, nor the engineer, but to the man. He will be to all of us always 'Robert Howell'."

Captain Howell was a member and an Elder of the First Presbyterian Church of Phillipsburg, and active in all church work. He was a member of the Knights of Pythias, Odd Fellows, Patriotic Order of Sons of America, a Mason, Knight Templar, Royal Arch Mason, and Shriner, and also belonged to the Elks.

Captain Howell was elected an Associate Member of the American Society of Civil Engineers on October 4th, 1905, and a Member on March 4th, 1913.

THOMAS WILLIAM JAYCOX, M. Am. Soc. C. E.*

DIED SEPTEMBER 8TH, 1919.

Thomas William Jaycox was born in Poughkeepsie, N. Y., on October 15th, 1851. He was the son of Thomas W. and Sarah Marie Rose Jaycox. At the age of twelve, he attended Putman's School for Boys at Poughkeepsie. After four years in Putman's School, he attended Fort Edward College in Northern New York. In 1869, Mr. Jaycox entered Cornell University as a student in Civil Engineering, and on April 18th, 1872, he was granted an honorable dismissal, having then completed his Junior year.

After leaving Cornell, Mr. Jaycox joined the Hayden Expedition as Assistant Topographer and was one of the members of that famous United States Geological Survey party during its work in Yellowstone National Park. In 1873, after leaving this survey, he accepted a position as Transitman with the New York Central and Hudson River Railroad on construction work. He left the railroad in September, 1874, to accept the appointment as Leveler in the Engineer Department, District of Columbia. During 1874 to 1877, he had charge of municipal construction, surveys, estimates, etc., in Washington, D. C., and from 1877 to 1880, he was also in charge, under Lieut. (now Brig.-Gen. (*Retired*)) R. L. Hoxie, U. S. A., M. Am. Soc. C. E., of the surveys, design, and construction of sewers in Washington.

In June, 1880, after serving more than five years at Washington, D. C., Mr. Jaycox moved to Leadville, Colo., and Colorado was his home from that time until his death. He had settled in Leadville with the rush that followed the discovery of carbonates in the district, and became the senior member of the firm of Jaycox, Goad, and Corning, one of the prominent firms of mining engineers in the early days of mining in that locality. Some twenty-four years of Mr. Jaycox's life work was spent in Leadville, and he was there recognized as one of the prominent mining engineers. His clientele represented many of the largest mining companies in the district, and his work for them covered the many and varied lines of mining, milling, power, and transportation problems.

In 1881, Mr. Jaycox was appointed City Engineer of Leadville. From 1892 to 1894, he served as an Alderman in the City Council and, later, was elected Mayor of Leadville, holding office for one term.

In 1903, when he was appointed Deputy State Engineer of Colorado, under Mr. L. G. Carpenter, then State Engineer, Mr. Jaycox moved to Denver. In 1905, he was appointed State Engineer of Colorado and held that office for four years.

In 1909, he resumed his private practice as Consulting Engineer with headquarters in Denver. His professional practice during the last ten years of his life was largely hydraulic, water supply, and irrigation work. He was retained as Consulting Engineer to design and supervise the construction of several reservoirs for irrigation and domestic supply. Mr. Jaycox also acted as expert witness and engineering adviser to legal counsel on numerous water litigation court cases in Colorado.

* Memoir prepared by Lyman E. Bishop, M. Am. Soc. C. E.

He was one of the members of the Board of Consulting Engineers for the City of Dayton, Ohio, appointed after the Dayton Flood, in 1913, to recommend measures to prevent the recurrence of that catastrophe. In this work he was associated with some of the most prominent engineers of the United States.

At the time of his death, Mr. Jaycox was retained by Lake County, Colo., to gather testimony for the Summit-Lake County Boundary litigation. It was while engaged on survey work on this boundary line that he was stricken with his fatal illness. Death resulted from dilation of the heart brought on by his strenuous survey work at high altitudes. He was ill only a few days, and his death came as a great surprise to his numerous friends.

Mr. Jaycox was married on January 27th, 1874, to Mary Anthony Dates, of Poughkeepsie, N. Y., who, with a son, survives him.

Mr. Jaycox was recognized as an engineer of high standing, and was highly respected by all with whom he came in contact. He was a man of high intellectual attainments, always a student in his profession, and of congenial personality. He was a family man, and his home and his profession made up his life.

He was a Charter Member of Leadville Lodge No. 51, A. F. and A. M., a member of the Colorado Scientific Society, a member of the Elks, and an Honorary Member of Tau Beta Pi.

Mr. Jaycox was elected a Member of the American Society of Civil Engineers on January 4th, 1882.

LORENZO MEDICI JOHNSON, M. Am. Soc. C. E.*

DIED NOVEMBER 28TH, 1904.

Lorenzo Medici Johnson was born in New York City on January 22d, 1843. His father was the Rev. Lorenzo Dow Johnson, and his mother's name before her marriage was Mary Burges. Until he was seventeen years of age, the boy lived with his parents in Plymouth County, Massachusetts, working on the farm and going to school.

In 1860, Mr. Johnson was appointed an Assistant in the United States Coast Survey, on the survey of Mobile Harbor and the projected Cape Cod Canal.

On September 27th, 1861, he was assigned to duty in the Pay Department of the Federal Army. He was with the Army of the Potomac at the capture of Yorktown by Gen. McClellan and during the Seven Days' Battle on the Peninsula, and also at the capture of Vicksburg. At the end of 1863, he was ordered to the Pacific Coast *via* the Isthmus and traveled through Oregon and the Territories of Washington and Idaho.

In 1867, Mr. Johnson was appointed a consular pupil and assigned to duty in Syria, where he served as Vice-Consul at Beirut, Jaffa, Jerusalem, and Damascus; he was promoted on July 13th, 1870, to be Consul General at Beirut. Realizing that, under the American system, this service would not become a certain career, he returned to the United States in 1871 with the intention of studying law, but was advised to study for a civil engineer's degree because of the rapid railway extensions then in progress. With this in view, he entered the Sheffield Scientific School of Yale University in 1871, and obtained his degree of Civil Engineer in 1875.

Immediately after graduation Mr. Johnson entered the service of the Keokuk and Des Moines Railway Company, passed through the grades of Engineer, Chief Engineer, Paymaster, and Assistant General Superintendent, and was appointed General Manager of the Cairo and St. Louis Railroad, holding that position until January 1st, 1881, when he was appointed Assistant to the President of the Pullman Palace Car Company, superintending the construction of the Town of Pullman. He had also been elected a Vice-President of the Cairo and St. Louis Railroad Company and held both positions until December 1st, 1883.

At this time he was appointed General Manager of the Mexican International Railroad Company, with headquarters at Ciudad Porfirio Diaz, Coahuila, Mexico. He also became General Manager of the Alamo, Coahuila and Rio Bravo Coal Companies, Superintendent of Construction of the American Development Company, and General Manager of the Coahuila and Durango Development Company, which positions he held for twenty years.

On January 1st, 1903, Mr. Johnson was made General Manager of the Railroad and Fuel Department of the American Smelting and Refining Company and General Manager of the Mexican Union Railway, El Carmen Railway, Aguas Calientes Railway, and Velardena Railway, with headquarters in the City of Mexico, Mexico.

On August 15th, 1904, he became President of the Pittsburgh, Shawmut and Northern Railroad, a position he held at the time of his death on November 28th, 1904.

* Memoir prepared by George B. Massey, M. Am. Soc. C. E.

Mr. Johnson was a member of the New England Society, the Society of the Sons of the American Revolution, the Sons of the Mayflower, the Western Society of Civil Engineers, and of numerous clubs.

He was married on April 22d, 1878, to Helen Wolcott Stewart, a daughter of General Hart L. Stewart, of Chicago, Ill. They had four children, Helen, Stewart, Dorothea, and Lesley, all of whom survived him.

Mr. Johnson was elected a Junior of the American Society of Civil Engineers on March 3d, 1875, and a Member on April 7th, 1880. He also served as a Director from 1897 to 1899.

NORMAN BENJAMIN KELLOGG, M. Am. Soc. C. E.*

DIED JANUARY 19TH, 1919.

Norman Benjamin Kellogg, the son of Charles and Jane (Burnell) Kellogg, was born at White Pigeon, Mich., on October 18th, 1843. On his father's side he was descended from Joseph Kellogg who came to America from Braintree, Essex County, England, about 1646, and settled first in Farmington, Conn., going thence to Boston, Mass., and finally settling in Hadley, Mass., and who served as an officer of the Colonial Army for more than thirty years, with the rank of Lieutenant.

His father, Charles Kellogg, moved from Sheffield, Mass., to White Pigeon, Mich., in 1831, where he engaged in business as a merchant and, later, as a merchant miller. In those days business was carried on mostly by water, and Charles Kellogg was also interested in the early navigation enterprises of the St. Joseph River. He was drowned in the spring of 1847 from a small steamer owned by his firm, leaving his widow with three small boys to bring up and educate.

Until he was twenty-five years old, and except for two winters under his brother's instruction at Ann Arbor, Mich., Norman Benjamin Kellogg remained in the Village of White Pigeon, preparing himself for college. In 1868, he entered Rensselaer Polytechnic Institute, at Troy, N. Y., and was graduated as a Civil Engineer with the Class of 1873. While at Rensselaer, he was elected a member of the Sigma Chi Fraternity.

From August, 1873, to July, 1874, Mr. Kellogg was employed by the Atlantic and Great Western Railway Company as Draftsman and Assistant to the Construction Engineer, with headquarters at Meadville, Pa. Subsequently, for a few months, he was engaged on the construction of bridge foundations for the Pennsylvania Bridge Company at Iona, Mich.

In March, 1875, he went to California where, until November of that year, he was employed as Assistant on field work and surveys for a narrow-gauge railroad. During the winter months he was engaged as Draftsman for private parties in San Francisco, Cal.

In April, 1876, Mr. Kellogg entered the service of the Central Pacific Railroad Company, as Chainman with a surveying party, and, later, as Rodman, Draftsman, and Leveler, until November, when he was made Transitman and also had charge of a line for the North Pacific Coast Railroad.

In January, 1877, he returned to the employ of the Central Pacific Railroad Company as Assistant Engineer on surveys and the construction of the Northern Railway, the San Pablo and Tulare Railroad, and the Benicia Branch Railway, and as Transitman on the Sacto Drainage District for the State of California.

From January, 1879, to March, 1880, Mr. Kellogg was engaged as Draftsman on location surveys for the Southern Pacific Company—the successor of the Central Pacific Railroad Company—in Arizona, New Mexico, and Texas, and from March to August, 1880, he was employed as Leveler and Transitman on construction work for that Company in those States. In August, 1880, he was promoted to be Assistant Engineer on this work and, later, was put in charge of construction on the Colorado Division, which position he retained until July,

* Memoir prepared from information on file at the Headquarters of the Society.

1883. In July, 1883, he was transferred to the Maintenance of Way Department of the Southern Pacific Company, and until January, 1891, served, successively, as Assistant Division Roadmaster, Division Roadmaster, and Resident Engineer in charge of the maintenance of way and water service of a division of from 800 to 1 000 miles of the main line.

Mr. Kellogg resigned his position with the Southern Pacific Company in 1891 and spent the next year or two traveling in Europe. On his return to the United States, he opened an office in San Francisco, Cal., for the private practice of engineering, where he was engaged on surveys, hydraulic projects, etc., until 1911, when he retired from active business. He spent a year or more visiting at his old home in White Pigeon, Mich., and then returned to California and settled in San Diego, where he resided until his death on January 19th, 1919. Mr. Kellogg was never married, and is survived by a niece, Mrs. Louise Maitland.

Mr. Kellogg was elected a Junior of the American Society of Civil Engineers on February 6th, 1878, and a Member on July 3d, 1895. He served as President of the San Diego Section of the American Society of Civil Engineers in 1917.

JOSEPH MOSS KNAP, M. Am. Soc. C. E.*

DIED JUNE 21ST, 1920.

Joseph Moss Knap was born in Brownsville, N. Y., on December 30th, 1837. He was graduated from Rensselaer Polytechnic Institute in 1858 with the degree of Civil Engineer. He was President of the Rensselaer Polytechnic Alumni Association in 1896-97-98, and of the New York Chapter of that Association for several years.

After graduating, Mr. Knap served for a short time in the Engineer Corps of the Pennsylvania Railroad and then went to Pittsburgh, Pa., where for two years he was Assistant Manager and Manager, respectively, of the Hiawatha Iron Works. The outbreak of the Civil War called him at once to the colors and on August 5th, 1861, he was commissioned First Lieutenant of Company L, 28th Regiment, Pennsylvania Volunteer Infantry. On October 5th, 1861, he was commissioned Captain of the Pennsylvania Light Battery, a unit of the 28th Regiment. This battery became famous in the Army of the Potomac as "Knap's Battery", and organizations did not become famous in that Army for nothing. It was engaged in many battles in the Peninsular Campaign, at Cedar Mountain, Antietam, Chancellorsville, Gettysburg, the Wilderness, and in many battles of the Wilderness Campaign. At Gettysburg, the battery was nearly annihilated, and two monuments to its men who died there, stand on that field.

On May 21st, 1864, Captain Knap was commissioned Major of the First Battalion of Artillery, Pennsylvania Volunteers. This battalion was organized for the defense of Washington and when the capital was threatened by Early's raid, in the summer of 1864, Major Knap was in command of the outer defenses. It will be noticed that he was not yet 27 years old. Two swords were presented to him, one by the men of the Battery and one by the City of Pittsburgh—a rare distinction. An organization known as "Knap's Battery, Pennsylvania Volunteers", still exists at Pittsburgh.

After the war Major Knap returned to Pittsburgh where, for seven years, he was in charge as Manager and part Proprietor of the Fort Pitt Foundry, established and owned by his uncle, Charles Knap. This foundry is historical for the famous Rodman guns which for the time put the United States in the lead of the world in heavy artillery, were cast there. Many 14 and 15-in. guns were cast, and the monitors were largely armed with them. They were also mounted in land fortifications. Several 20-in. guns were also cast and mounted in the forts of New York Harbor, at Fort Monroe, and possibly elsewhere. In 1872, Major Knap went to Constantinople to sell guns to the Sultan of Turkey, but was not successful. The Fort Pitt Foundry also made heavy duty pumping engines for city water-works.

For five years, Major Knap was Superintendent and part owner of the Charlotte Furnace and Coal and Iron Mines in Westmoreland County, Pennsylvania. In this capacity his war experience and aptitude came into play. A strike was beginning in the region and threats were made to burn the company store. Major Knap's men would not join the strikers, but organized a guard to protect the company's property, which was not molested.

* Memoir prepared by Palmer C. Ricketts and Henry G. Prout, Members, Am. Soc. C. E.

In 1878, Major Knap gave up engineering and allied work, removed to New York City, and engaged in commercial business, in which he spent the remainder of his active life. In 1901, his health failed, and he and his wife retired to their country home at Catskill-on-Hudson, and there his remaining years were passed quietly, until his death on June 21st, 1920. During the time that he was Treasurer, Major Knap made frequent visits to New York on the business of the Society.

Major Knap's brief and brilliant military career was the logical expression of his character. He was brave, quick to decide, determined, and strong. He was high-spirited and had high ideals. He was kind and friendly, broad and generous. These are the foundation qualities of a leader, and that Major Knap was; and he was a charming companion and a loyal friend. He was always interested in the Engineering Profession and fond of the associations that it brought to him. He was a diligent attendant at the meetings and conventions of the Society. In recent years, he had taken little part in technical papers or discussions, but was interested and influential in affairs of organization, finance, and administration, and in that respect was one of the builders of the Society during long years of great importance in its development.

Major Knap was a keen member of the Military Order of the Loyal Legion, and this association gave him great enjoyment. He was also a member of the Engineers' Club and the Colonial Club.

In the fall of 1864, Major Knap was married to Miss Sophie Hale Day, of Catskill, N. Y. Three children survive them, Edgar Day Knap, Assoc. M. Am. Soc. C. E., and Mary Knap of Catskill, and Joseph Day Knap of New York City.

Major Knap was elected a Member of the American Society of Civil Engineers on April 5th, 1882. He served as a Director from 1894 to 1896, and as Treasurer from 1900 to 1912.

JOHN LANGTON, M. Am. Soc. C. E.*

DIED FEBRUARY 6TH, 1920.

John Langton was born at Toronto, Ont., Canada, on August 29th, 1857. He was a student in the Engineering Course at Owens College, Manchester, England, from 1873 to 1875, holding the Ashbury Engineering Scholarship.

In 1876-77, he served as an Apprentice in the drafting room and machine shop of the Portland Company, at Portland, Me., and in 1879-80, in the machine shops of the Northern Railway, at Toronto. In 1881-82, he was employed as Mechanical Draftsman by the Brown and Sharpe Manufacturing Company, at Providence, R. I.

In June, 1882, Mr. Langton joined the forces which Thomas A. Edison was then organizing for the introduction of his electric lighting system, and was appointed Mechanical Engineer for the Electric Tube Company, in New York City. When, in 1885, this Company was consolidated with the Edison Machine Works, he was made Superintendent of the Brooklyn shops. He was then transferred to Schenectady, N. Y., where Mr. Samuel Insull was concentrating all the Edison manufacturing work in the plant now operated by the General Electric Company. At first, Mr. Langton served as Superintendent of the Testing, Armature Winding, and Electric Tube Departments, but a little later he became Chief Engineer of the Edison Machine Works. This led to his selection, in May, 1889, as General Manager of the Canadian Edison Manufacturing Company, at Sherbrooke, Que., Canada, and when that concern was consolidated, in 1890, with others to form the Edison General Electric Company, he was made Manager of the Canadian Works. The rapid growth of the Edison system in the Dominion led to the removal of these works, in May, 1891, to much larger shops at Peterboro, Ont., Canada, where the plant has since remained, bearing to this day the imprint of Mr. Langton's remarkable engineering and constructive skill. He was tired out, however, by such continuous and strenuous application, and, in June, 1892, resigned his position with the Edison Company to take up private practice. This, though extensive, was limited at first largely to his native land, and Mr. Langton had offices at Toronto until 1895, when his engagements in the United States as a Consulting Engineer, particularly in the mining field, necessitated the establishment of offices in New York City.

Mr. Langton's distinctive electrical career closed about this time, but from it he retained many friendships and pleasant memories, and he was proud to have been from its foundation, a member of that select little brotherhood known as the "Edison Pioneers". Already a member of the American Institute of Electrical Engineers to which he was elected in June, 1888, he became a member of the Canadian Society of Civil Engineers in December, 1892. He also served very acceptably in 1896, 1897, and 1898, as an Examiner for post-graduate degrees of M. E. and E. E., for the University of Toronto.

Mr. Langton's work in the mining field constitutes a chapter by itself. From 1900 to 1904, he was engaged in the design and construction of the entire smelting plant for the Copper Queen Consolidated Mining Company, at Douglas, Ariz.

* Memoir prepared by Thomas Commerford Martin, Esq.

This plant consisted of five 300-ton blast furnaces, four converters, a complete power plant, repair shops, etc. Of this work, a well known expert has said: "To my mind this was Mr. Langton's masterpiece, as the capacity of the plant has been practically trebled; and as it is now, no one can see where additions have been put in." In 1904-05, came the installation of the generating plant, transmission line, and rotary converter sub-station for the mining plant of the Moctezuma Copper Company at Nacozari, Sonora, Mexico. In 1906-07, Mr. Langton installed approximately 450 beehive under-flue coke ovens at the plant of the Dawson Fuel Company, at Dawson, N. Mex., where the waste heat gasses from the ovens are used to generate steam. This work included a complete power plant, comprising four 400-kw., 25-cycle, 2 300-volt, engine-driven generators, four rotary sub-stations, electric pumping plant, etc., for the coal mines. In 1908, jointly with Mr. C. Legrand, he carried out the design and construction of a complete power plant of three 1 000-kw. turbines for the Moctezuma Company, and a combined turbine and compressed air plant for the Copper Queen Mines at Bisbee, Ariz. In fact, Mr. Langton devoted practically all his time from 1899 to 1908, to the enterprises which form the present Phelps-Dodge Corporation.

In 1908, he went back to private practice, and since that time he had remodeled the power plant of the Cananea Consolidated Copper Company and supervised the design of the power plant of the Inspiration Copper Company, comprising two 6 000-kw. turbines and several blowing engines. In conjunction with Mr. F. L. Antisell, he had designed a large addition to a Raritan copper refinery. Another interesting piece of work which he also finished was that at Morenci, Ariz., including a 1 000-h.p., gas-generating plant and a system of gas distribution to scattered gas engines, 800 h.p. all told, 200 h.p. of which drive polyphase generators in parallel for power distribution to induction motors.

Death came all too soon, but it found Mr. Langton as he would have wished, on the field of duty, at Inspiration, Ariz., where he was engaged on the design of additions to the plant. The news that he had passed away on February 6th, 1920, was a terrible shock to a host of friends who had learned to love and admire this quiet, reserved engineer, who was essentially a cultivated gentleman of the old school.

Mr. Langton was a student, scholarly but not pedantic, a judicious lover of rare books and choice vintages, not a bon-vivant, but a good companion; always successful in his work, but, somehow, one who would hardly have been singled out for intimate participation in the swift, startling developments of the formative days of electrical engineering, or the rough-and-tumble application of scientific advances to the crude conditions of frontier mining. Through it all, however, he was a man among men, never losing his poise, and always wearing a subtle air of distinction. John Langton "made good."

Mr. Langton was elected a Member of the American Society of Civil Engineers on May 2d, 1900.

LOUIS JULIAN LE CONTE, M. Am. Soc. C. E.*

DIED MAY 11TH, 1920.

Louis Julian Le Conte was born at Savannah, Ga., on March 3d, 1845. The Le Conte family is of Huguenot origin, and is descended from Guillaume Le Conte, who was born in Rouen, France, on March 6th, 1659. Sometime during the last decade of the Seventeenth Century he came to America, and died in New York, in February, 1710. The name, Le Conte, was continued through Pierre, his second son. The family soon went South from New York to Liberty County, Georgia, where they developed a large estate and, in the Revolutionary struggle, took very prominent parts.

Louis Julian Le Conte, as he grew up, attended the schools of his native city and entered the University of South Carolina. It was natural that, in the Civil War that came in the course of the development of the country, the Le Contes should espouse the cause of the South, and at the early age of 17 Louis Julian Le Conte, leaving the University, joined the Confederate Army and, with the same devotion and faithfulness that marked his entire career, went through to the end. After the war, he returned to the University of South Carolina, in which his father and uncle were Professors, and was graduated in 1868.

In 1869, the family went West to California. The father, John Le Conte, became Professor of Physics, Industrial Mechanics, and Physiology in the University of California, serving as such from 1869 until his death in 1891; he was also President of the University from 1876 to 1881. The uncle, Joseph Le Conte, was Professor of Geology and Natural History in the same institution from 1869 to the time of his death on July 6th, 1901, in the Yosemite Valley. Both his father and uncle had published many scientific works, and numerous points of geographical interest in California are named for them.

Young Le Conte, after an experience of two years from 1869 with the New Almaden Quicksilver Mines and on surveys, in 1871, at the age of 26, well equipped for his life's work, entered the service of the United States in the Corps of Engineers of the Army, under Gen. B. S. Alexander. Later, he became the invaluable Assistant of the late Col. George H. Mendell, M. Am. Soc. C. E., and served forty working years of his life on river and harbor improvements and on the fortifications of California, in the planning and construction of works of the first importance to this and future generations. This work included Oakland Harbor; the Sacramento and San Joaquin Rivers; Mormon Slough from the City of Stockton to its entrance into the San Joaquin River; and the survey of San Francisco, San Pablo, and Suisun Bays, in all, a body of water approximating 700 sq. miles in area; and other works of lesser magnitude.

After the retirement of Col. Mendell from the Army in 1895, Mr. Le Conte continued as Assistant Engineer under the former's successors, the late Charles R. Suter, Brig.-Gen., U. S. A. (*Retired*), and Charles E. L. B. Davis, Brig.-Gen., U. S. A. (*Retired*), W. H. Heuer, Col., U. S. A. (*Retired*), and John Biddle, Maj.-Gen., U. S. A., all members of the Society.

* Memoir prepared by Horace D. Gates, M. Am. Soc. C. E.

A mere incomplete, chronological statement of the works on which Mr. Le Conte was engaged, gives only a faint idea of the magnitude and quality of the results of his labors; it must be supplemented by a contemplation of the fine character of the man, as shown by the affectionate tributes to him of a few of his friends and co-workers.

Col. Heuer, under whom he served from 1897 to 1907, writes of his "excellent service", and that he "was always interested in and loyal and efficient at every work on which he was engaged. His practically continuous work was the local charge of operations in Oakland Harbor, from its inception to the time of his resignation in July, 1911. Very much of the success attending the making of a harbor for Oakland must be attributed to Le Conte."

Otto von Geldern, M. Am. Soc. C. E., contributes the following:

"There is no one for whom I had a greater respect and who stood higher in my estimation, as a man and an engineer, than my dear old friend L. J. Le Conte. It was a pleasure to know him. I have never known in all my life a more honest character. He was scrupulously honest in the smallest detail of everything that he did. He was a man of many talents and the most scholarly engineer I have ever known. I served with him for several years, and our relations were not marred for a moment by an unpleasantness. He never gave a man an opportunity to stir up anything of the kind. He was actuated by the highest motives and absolutely unselfish.

"If there is anything that I can do to perpetuate his memory, it shall be a sacred duty with me."

C. E. Grunsky, M. Am. Soc. C. E., testifies as follows:

"Mr. Le Conte impressed me greatly the first time I met him in Sacramento in 1879. He and I had a long discussion of river problems. In subsequent years we met occasionally and I derived pleasure from every meeting. He was so kindly and well disposed toward every one and so well informed and so careful and painstaking in the work with which he was entrusted that he could not but command the admiration of those with whom he came in contact. The faithfulness with which he served the United States is possibly without a parallel. At a small salary not commensurate with the value of his services he was an Assistant Engineer with the U. S. Army Engineers in charge of such important work as the improvement of Oakland Harbor. After 40 years of service, faithful and able, he retired, dropping back into private life without pension or other reward than the knowledge of having served his country long and faithfully."

With this tribute, Mr. Grunsky enclosed a copy of Mr. Le Conte's letter of resignation, kindly furnished by the late Capt. de Merritt, U. S. Corps of Engineers, as follows:

"JULY 6, 1911.

"COL. JOHN BIDDLE,
"Engineer Corps, U. S. Army,
"San Francisco, Cal.

"DEAR SIR.—On June 28th, 1911, my eyesight had somewhat improved by 60 days rest which you were kind enough to give me. I consulted several oculists and they all told me that in the present condition of my eyes it would be extremely hazardous to make any practical use of them for fear of a relapse, resulting in general blindness. Of course, I could not think of courting such a terrible calamity, therefore, I would respectfully beg to be excused from attempting it.

"I feel that my days of usefulness are over and my career of 40 years service in the Corps of Engineers has come to a close.

"(Signed) L. J. Le Conte."

Marsden Manson, M. Am. Soc. C. E., writes as follows:

"It was my privilege to form an early acquaintance with Mr. Le Conte, an acquaintance which soon ripened into a warm admiration of his personal and professional worth. He was at the time in direct charge of the improvement of the Oakland, Cal., Estuary into a harbor of great usefulness. In this work he supervised the building and successful operations of the first hydraulic dredger. This dredger was designed by A. W. von Schmidt, and was used continuously for some years.

"Mr. Le Conte had the charm of manner and courteous demeanor of the Southern gentleman. Born of Huguenot ancestry, he came to California with his distinguished father and uncle, who served in the faculty of the University of California.

"Mr. Le Conte's contribution to river and harbor work was more noted in official reports than in professional papers. His broad acquaintance with the problems of water supply is attested in many professional papers and discussions.

"Coupled with high professional attainments, was a nobility and gentleness of character which his friends will always cherish, and which leaves an honored memory to his family."

Others, not easily reached at the time, would doubtless have responded in terms similar to these testimonials. In all the long years of anxiety and pain which Mr. Le Conte endured, he never uttered a word of complaint. How characteristic this was, those who knew him will realize.

The compiler of the above records would echo the sentiments therein expressed, as the climax of a friendship of more than forty-two years.

On July 20th, 1880, Mr. Le Conte was married to Miss Mary W. Harmon, of Oakland, Cal., who survives him, together with five daughters and one son.

He was one of the first members of the American Water Works Association.

Mr. Le Conte was elected a Member of the American Society of Civil Engineers on April 4th, 1877.

GEORGE LEIGHTON, M. Am. Soc. C. E.*

DIED SEPTEMBER 12TH, 1920.

George Leighton was born on May 2d, 1859, at Waverly, Pa., the son of Andrew and Frances (Sisson) Leighton. He was of Scotch descent, although his parents were born in America. His father was engaged during early manhood in literary activities and was known in publishing circles in New York City and Albany, N. Y. His home in Pennsylvania, where he had a farm, was a local center of intellect and culture, under which influences the son received his first impressions.

After preparatory education at Waverly and Keystone Academies, Mr. Leighton entered Lehigh University with the Class of 1883. While at Lehigh, his literary bent led him to the editorship of *Epitome* and *Lehigh Burr*, the latter of which he took part in founding. He was a Charter Member of the Eta Chapter of the Psi Upsilon Fraternity at Bethlehem.

In 1881, he left college to start an engineering career. Like many other young engineers of that period, who have since become well known, Mr. Leighton began as a Rodman in railroad construction. The next five years found him steadily advancing in various positions and gaining valuable experience. From 1886 to 1888, he was Assistant Engineer on the construction of Washington Bridge across the Harlem River, in New York City, of which the late William R. Hutton, M. Am. Soc. C. E., was Chief Engineer, and the late Alfred Noble, Past-President, Am. Soc. C. E., and the late John Bogart, M. Am. Soc. C. E., successively, Resident Engineers.

From 1888 to 1893, Mr. Leighton was engaged with interests financing and building railroads in the South and Southwest, during which time, as Chief Engineer, he built the East Shore Terminal, at Charleston, S. C., and made examinations of other constructions and reports on projects. Between 1895 and 1901 he was in private practice in Scranton, Pa., engaged in sewerage and water-works construction and in railroad and mine surveys. Following that, he was in charge of bridge erection on a part of the Delaware, Lackawanna and Western System.

In 1905, he became Resident Engineer of a part of the Pennsylvania Railroad Tunnels under the East River, of which work the late Mr. Alfred Noble was Chief Engineer. When that work was nearing completion, Mr. Leighton left to take charge of an important part of the Delaware, Lackawanna and Western Cut-Off in New Jersey, where he remained until 1911. The last four years of his professional career were occupied with the construction and operation of water-works and sewers in Lyons, N. Y.

Besides being a broad and practical engineer, Mr. Leighton was a close student and was well grounded in engineering theory in general. The first textbooks published by the International Correspondence Schools on the subject of Civil Engineering were prepared by him. He was an enthusiastic worker, never sparing effort or hours, and imbued those about him with a like spirit. His professional relations were characterized by honesty of the most direct and fearless kind. Quick to recognize the right course in any situation, he would pursue it promptly and

* Memoir prepared by Frederick C. Noble, Paul G. Brown, and St. John Clarke, Members, Am. Soc. C. E., and David Fox, Esq.

positively and no selfish consideration could possibly give him pause. Not for an instant would he tolerate any form of deception. No one could doubt his courage or determination to make his work of the very best. To his subordinates, though not lenient, he was fair and considerate. Any engineering assistant whom he observed taking especial interest in his work would be certain of helpful explanation or kindly advice.

His personal side represented a rare combination of splendid qualities. The greatest sincerity and earnestness marked his manner and every action. At first meeting he might appear distant, but this impression would vanish with fuller acquaintance. As a friend Mr. Leighton was loyal, sympathetic, and constant. His generous and unselfish impulses made him appear almost quixotic at times. It is related of him that when certain railroad work of which he had charge failed for lack of funds, finding that the laborers had not been paid, although he had been, he distributed what he had among them, keeping only enough to take him back home.

He had a keen appreciation of Nature, was well informed regarding the arts, and enjoyed the best in music and literature. His conversation was entertaining, instructive, forceful, and always to the point. Religious by inclination and early influences, he always retained connection with his home church.

In 1888 he was married to Theodora, daughter of Theodore Ross, of Tunkhannock, Pa., a lineal descendant of John Alden, the Pilgrim.

His death occurred on September 12th, 1920, after several weeks' illness which, fortunately, left him oblivious of its course. He is buried near his birthplace at Waverly. His widow, two sons, and two daughters survive him. The eldest son, James, is a civil engineer and served overseas during the World War, with an engineer regiment. The eldest daughter, Margaret, is head of the Physical Education Department of the Young Women's Christian Association, of Louisville, Ky. The younger son and daughter are attending school.

Mr. Leighton was elected a Member of the American Society of Civil Engineers on October 3d, 1906.

CHARLES ARTHUR LINDBERY, M. Am. Soc. C. E.*

DIED MAY 27TH, 1918.

Charles Arthur Lindbery, one of the members of the Society who gave his life to the service of his country during the World War, was born in Albany, N. Y., on August 25th, 1878. When he was yet a boy, his parents, Albert Lindbery and Carolina Schyberg Lindbery, responded to the lure of the West and moved to the State of Washington. Young Lindbery completed his secondary education at, and was graduated from, the Whatcom High School, at Bellingham, Wash., in 1898. He entered the University of Washington in the fall of 1898, at the same time the writer went there as a member of the Faculty.

Mr. Lindbery's college work showed him to be a normal young American. He did well in his classwork and maintained an active interest in college activities. His greatest outside interest, perhaps, was in the Student Cadet Corps, in which he became Captain of his Company. This early interest in military work and the proficiency gained was unquestionably a factor in bringing him into the Army and in developing him into an efficient officer.

He started his engineering experience on public land surveying immediately on leaving college. After a year, Mr. Lindbery became Assistant City Engineer of his home town, Bellingham, Wash., and remained in that position for four years, under gradually increasing responsibilities.

In 1906, he began a railroad experience as Transitman on the picturesque location of the Oregon and Washington Railroad, up the Deschutes Canyon, from the Columbia River to Central Oregon. Later, he was made Chief Transitman for the same road on the Seattle Tunnel and on other work.

In 1908, his home community again called Mr. Lindbery as County Engineer of Whatcom County. For the next five years, in that position, he had full charge of a large amount of road and bridge construction and river protection work. He was also engaged on private work during that time, as Chief Engineer on five large drainage systems.

In 1913, he was called to the City of Sumas, Wash., for a short time, to direct its paving and sewer work, and also served as Consulting Engineer to Mr. C. H. Kiehl on the Great Northern Concrete Key Dock, at Vancouver, B. C., Canada.

These combined duties put Mr. Lindbery under a severe strain which made it necessary for him to give up active work for the time being, and he repaired to San Diego, Cal., to recuperate.

When his health permitted, he again went back to his old home as County Engineer, and was granted a leave from that position to enter the military service. He was commissioned Captain in the Engineer Reserve Corps, on September 21st, 1917, and ordered to active duty at the Engineer Officers' Training Camp, Camp Lee, Va., on December 19th, 1917. On March 5th, 1918, Captain Lindbery was assigned to the 601st Engineers, at Camp A. A. Humphreys, Va., which regiment was later transferred to Camp Laurel, Md. On May 21st, 1918, he reported at Camp Lee, Va., for duty as student at the Engineer Officers' Training Camp,

* Memoir prepared by Almon H. Fuller, M. Am. Soc. C. E.

but subsequent orders were issued relieving him from duty at that post and directing him to proceed to Camp A. A. Humphreys. Here he was taken ill, following a chill and a broken arm which he received from going into the water to help a comrade, and was removed to the Base Hospital at Camp Lee, Va., where he died of broncho-pneumonia on May 27th, 1918. That a career so potential in engineering and military possibilities should be suddenly ended by disease seems to be one of the ironies of fate.

Captain Lindbery was married twice, in 1907, to Miss Ada Wilson who died in 1910, and, in 1916, to Miss Lottie Larson, who survives him. His mother died in 1910, but his father is still living. He was blessed with no children.

Captain Lindbery was elected a Member of the American Society of Civil Engineers on October 1st, 1913.

ARCHIBALD BYRON LUEDER, M. Am. Soc. C. E.*

DIED AUGUST 21ST, 1920.

Archibald Byron Lueder was born at Nanticoke, Pa., on August 26th, 1876. He entered Cornell University in the fall of 1895 and was graduated from the College of Civil Engineering in 1899, at which time he started his professional career as Timekeeper on construction work with the Berlin Iron Bridge Company of East Berlin, Conn.

After serving for a time as Foreman of Erection with this company, Mr. Lueder was sent, in the spring of 1901, by the American Bridge Company, as Agent and Assistant Engineer on the construction of twenty-eight viaducts and bridges on the Uganda Railway in British East Africa, Port of Killandini. This interesting construction work was carried out under unusually difficult conditions. The railway was constructed through an uncivilized section of East Africa, and most of the work was done with native laborers who had had no experience in steel construction work.

On the completion of this work, Mr. Lueder returned to America in the spring of 1903 and was employed by the American Bridge Company as Assistant Engineer on the construction of a number of bridges for the Wabash Pittsburgh Terminal Railroad, between Mingo Junction, Ohio, and Pittsburgh, Pa. From the fall of 1904 to February, 1909, he was Assistant Engineer with the same company in charge of the erection of steel work in the New York District. Some of this work included the Rolling Lift Bridge, Vernon Avenue, Brooklyn; Belmont Race Track Grandstand, Belmont Park, Long Island; Waverly (N. J.), warehouse for the Carnegie Steel Company; Hackensack River Draw-Bridge for the Erie Railroad; replacing the two-span bridge over the Central Railroad Yards at Jersey City, N. J., for the Lehigh Valley Railroad; and the Ferry House at the foot of West 23d Street, New York City, for the Delaware, Lackawanna and Western Railroad.

In February, 1909, Mr. Lueder joined the Merrill Ruckgaber Company, with which he continued until December, 1912. The work constructed under his supervision during this period included steel and concrete highway bridges and a concrete sewer on Broad Street, Stapleton, Staten Island, some difficult sewer construction on Malta Street, Canarsie, Brooklyn, and the construction of the Evitts Creek Reservoir and Dam at Cumberland, Md., which included nine miles of wood-stave pipe.

In December, 1912, Mr. Lueder was appointed Superintendent for the Arthur McMullen, Snare and Triest Company, on the construction of the 125th Street Section of the Lexington Avenue Subway, and after this work was well under way, he was sent by the Snare and Triest Company to Coquimbo, Chile, in charge of the construction of ore docks for the Bethlehem Steel Company.

Returning from Chile during the spring of 1915, he was first employed by the Jarrett Chambers Company, and, later, by the Phœnix Construction Company, on construction work at the power plant, at Omaha, Nebr.

* Memoir prepared by E. J. Moore and Alexander Thomson, Jr., Members, Am. Soc. C. E.

Mr. Lueder was connected with the Electric Bond and Share Company as Manager of construction work from the summer of 1916 until early in 1919, and among the works of which he had charge during this time were the following: The installation of a 15 000-kw. steam turbine at Omaha, Nebr., together with the construction of a condensing water intake of about 100 000-kw. capacity on the Missouri River, involving deep coffer-damming; the installation of a 10 000-kw. steam turbo-generator at Wichita, Kans., with the condensing equipment, boilers, spray pond, etc.; and the installation of a 15 000-kw. steam turbo-generator at Fort Worth, Tex., with the condensing equipment and boilers; and the construction of a condensing water intake and tunnel on Trinity River.

During 1919, Mr. Lueder devoted his time to the development of oil properties near Fort Worth, Tex., having been President of the Phœnix Oil Company which he organized and managed until it was necessary for him to give up his work because of ill health which developed early in 1920.

In addition to being very competent in directing construction work, Mr. Lueder was a genius in overcoming difficulties and possessed fine human qualities which enabled him to build up an organization by gathering together a loyal and efficient construction force.

He was married at Morristown, N. J., in June, 1903, to Mary Rosevear, who, with one son, Reginald, survives him.

Mr. Lueder was elected a Member of the American Society of Civil Engineers on May 31st, 1910.

ALEXANDER McCLURE LUPFER, M. Am. Soc. C. E.*

DIED FEBRUARY 3D, 1920.

Alexander McClure Lupfer was born at Blaine, Pa., on September 17th, 1855. He was the son of Samuel Lupfer and Matilda McClure Lupfer.

His early education was obtained in the public schools at Newville, Pa., to which place his parents moved soon after his birth. Later, he attended and prepared for college at the State Normal School, at Millersville, Pa. He entered Lafayette College, at Easton, Pa., in 1876, and was graduated from the Civil Engineering Department thereof in 1880.

Mr. Lupfer spent the first year after his graduation as Chainman and Instrumentman on the New York, West Shore and Buffalo Railroad. Like so many of the young engineers of that period, he had his eye on the great West where many preliminary and location surveys were being made, and construction work was being started for several large railroad projects. Accordingly, in the summer of 1881, he went West and from July of that year until early in 1883 he was Transitman and Assistant Engineer on new lines for the Denver and Rio Grande Western Railway.

From May, 1883, to May, 1884, Mr. Lupfer was engaged as Transitman and Assistant Engineer for the Oregon Railroad and Navigation Company, in connection with changes of grade and alignment along the Columbia River and on the construction of what was then called the Baker City Branch, now a part of the main line between Umatilla and Huntington, Ore.

He was an Assistant Engineer for the Northern Pacific Railway from July, 1884, to June, 1885, on the construction of the main line along the Yakima River, Washington.

Following this Mr. Lupfer was for a few months in private engineering practice at Portland, Ore., during which time he made the original survey of the Ilwaco Railway in Southwestern Washington.

From November, 1885, to June, 1887, he was Assistant and Division Engineer on the location and construction of the Minnesota and Northwestern Railroad, now a part of the Chicago Great Western Railroad in Minnesota and Iowa.

He was employed as Division Engineer on location and construction on the Illinois Central Railroad, from July, 1887, to January, 1888, and as Assistant Engineer on location and construction of new lines for the Union Pacific Railroad, and the Denver and Rio Grande Western Railroad from May, 1888, to May, 1890.

From May to August, 1890, Mr. Lupfer was engaged as Assistant Engineer at the end of the track in charge of bridges and track-laying on the Duluth and Winnipeg Railroad, now a part of the Great Northern Railway in Northeastern Minnesota.

In August, 1890, he entered the employ of the Great Northern Railway under the late E. H. Beckler, M. Am. Soc. C. E., on the location and construction of its Pacific Extension from Havre, Mont., to Seattle, Wash., remaining on that work until its completion in the early part of 1893. He had charge of very

* Memoir prepared by A. H. Hogeland, M. Am. Soc. C. E.

important location and construction work on this Pacific Extension, including the mountain sections on the east and west slopes of the Cascades, and the Switch Back, about twelve miles in length, through Stevens Pass, over the Cascade Tunnel.

From February, 1893, to June, 1896, Mr. Lupfer was Assistant Engineer on Maintenance of Way on the Great Northern Railway, with headquarters, first, at Spokane, Wash., and, later, at St. Paul, Minn.

From June, 1896, to October, 1900, he was employed as Engineer in charge of parties engaged on the surveys and construction of new lines and revisions of old lines for the Great Northern Railway in Minnesota, North Dakota, Montana, and Washington. During this period he located several lines in the Columbia and Okanogan Valleys, Washington, including the line afterward built from Wenatchee to Oroville; the change of main line along the Pacific Coast between Belleville and Bellingham, Wash.; and the Park Rapids and Leech Lake Line in Northern Minnesota.

Mr. Lupfer then returned to the East for a short period, and was engaged, from November, 1900, to May, 1901, as Chief Engineer of the Erie and Wyoming Valley Railroad, in Pennsylvania, now a part of the Erie Railroad.

He then resumed work for the Great Northern Railway, and from June, 1901, to December, 1904, was Engineer in Charge of Surveys and Construction of several important lines in Montana and British Columbia, including the change of main line, about seventy miles in length, through the Cabinet Mountains, Columbia Falls to Rexford, Mont.; the Crow's Nest Southern Line into the coal mines at Fernie, British Columbia; and the location of the Flathead South Line along the east side of Flathead Lake, connecting the Great Northern Railway at Somers with the Northern Pacific Railway at Dixon, Mont., which has not as yet been built.

In January, 1905, Mr. Lupfer became Chief Engineer of the Spokane and Inland Empire Railroad, continuing in that position until May, 1911, during which time he located and constructed the lines of that System, extending from Spokane to Palouse and Colfax, Wash., and Moscow, Idaho.

From September, 1911, to January, 1912, he was engaged on an examination of mining properties and proposed railroad lines in connection therewith in Brazil for American and English interests.

He made a trip to Spain with John F. Stevens, M. Am. Soc. C. E., in connection with proposed railroad improvements in that country, in the early part of 1912, and from September to December, 1912, he was engaged as Assistant Engineer on grade separation work, then just being started for the Northern Pacific Railway, at Spokane, Wash.

On January 1st, 1913, he was made Chief Engineer of the Spokane, Portland, and Seattle Railway, the Spokane and Inland Empire Railroad, and Oregon Trunk Railway, with headquarters at Portland, Ore. Later in the same year, he was appointed also Chief Engineer of the Oregon Electric Railway, United Railways, at Portland, and the Pacific and Eastern Railway. He continued in these positions until his death, which occurred at his home in Portland, Ore., on February 3d, 1920.

Mr. Lupfer belonged to that fine body of engineers who blazed the trails of the great transcontinental railways. His natural ability and his large and varied

experience were such that he became one of the recognized great locating and construction engineers of the Northwest. His skillful and methodical work was an inspiration to all who labored with him, and by his genial, kindly, and considerate manner he won the hearty and enthusiastic support of all his subordinates. He was a man of strict integrity, high ideals, and was always held in the greatest esteem by his employers and associates. He felt at all times the great responsibility placed on him in his large construction undertakings; in securing the best location; in properly planning all the works; and in dealing justly with both his employer and the contractor. His sound judgment in dealing with contractors was well recognized by his employers and others, and he was frequently selected to arbitrate matters in dispute under other engineers.

Mr. Lupfer was especially interested in the young men of the Engineering Profession, and many can testify to the cheerful word, the wise counsel, and helping hand extended when the way seemed very dark.

In his home life, and in the communities in which he lived, he was the kindly Christian gentleman, taking an active part in civic and church affairs—loved and honored by all.

Mr. Lupfer was a member of the First Presbyterian Church and the University Club of Portland, Ore.; of the Sons of the American Revolution, the Phi Delta Theta Fraternity, the York and Scottish Rite Masons, and the Mystic Shrine.

He was married on December 25th, 1893 to Miss Lottie D. Eckbert, of Curwensville, Pa., who, with their only child, Elizabeth, survives him.

Mr. Lupfer was elected a Member of the American Society of Civil Engineers on November 1st, 1905.

MICHAEL LEHANE LYNCH, M. Am. Soc. C. E.*

DIED DECEMBER 26TH, 1920.

Michael Lehane Lynch was born at Cork, Ireland, on April 8th, 1844.

After coming to the United States, Mr. Lynch was employed during part of 1871, and also in 1873 and 1874, as an Assistant Engineer in the United States Engineer Corps on river and harbor works in Louisiana and Texas.

Leaving the Government Service, he spent part of 1871 and 1872, and also from 1875 to 1877, as Assistant Engineer and Draftsman with the Texas and Pacific Railway Company, on the layout of yards and the construction of roundhouses, machine shops, etc. In 1873, he was engaged as Assistant Engineer on the construction of a system of sewerage for Galveston, Tex., and as Assistant Engineer and Draftsman for the International Railroad Company.

In 1878, he engaged in private practice as an Engineer and Architect, with an office in Galveston, Tex., which practice he continued through 1879. During these two years he also served as City Engineer of Galveston.

In 1879, Mr. Lynch returned to railroad engineering and served until 1882 as Topographer, Division Engineer, and Engineer in charge of masonry construction, with the Gulf, Colorado, and Santa Fé Railway Company.

From 1882 to 1884, he was again engaged in private practice as a Consulting Engineer, and during this time he designed and superintended the construction of various water-works systems, including those for Temple, Belton, Victoria, and Brownwood, in Texas. He also made an examination and report on the water supply for Galveston, Tex., and constructed several highway bridges across the Bosque, Concho, and Colorado Rivers, in Texas.

In 1884, he returned to the employ of the Gulf, Colorado, and Santa Fé Railway Company as Chief Engineer in charge of surveys and construction, in which connection he made extensive reconnaissances through Texas and Indian Territory, remaining in this position until 1887.

In 1887, and part of 1888, Mr. Lynch was employed as Division Engineer in charge of location and construction on the Shreveport Branch of the St. Louis, Arkansas and Texas Railway (now part of the St. Louis Southwestern System), from Lewisville, Ark., to Shreveport, La.

On the completion of this work in the latter part of 1888, he entered the firm of Hurley, Tierney, and Lynch, Engineers and Contractors, with headquarters at Fort Worth, Tex. As Engineer of this firm, Mr. Lynch designed the remodeling of the San Angelo Water-Works, and was engaged on the power development of the Concho River. He also designed the water supply system and water power development for the Llano Furnace and Improvement Company, and served as Chief Engineer on the construction of the Fort Worth and Albuquerque Railway (which was never completed). He also located, and his firm constructed, the Fort Worth and Rio Grande Railway (now part of the St. Louis-San Francisco System) from Granbury to Brownwood, Tex. Afterward, he had charge of the relocation and grade revision of the Minneapolis and St. Louis Railway.

* Memoir prepared from information furnished by C. D. Purdon, M. Am. Soc. C. E., and on file at the Headquarters of the Society.

On October 1st, 1899, Mr. Lynch was appointed Chief Engineer of the St. Louis Southwestern Railway Company and as such had charge of relocation and grade reduction on a great part of the road. Owing to failing health, due to overwork and grief over the death of his wife, Mr. Lynch resigned his position in April, 1910, and retired from active work. He spent his remaining years in Jackson, Miss., making his home with his brother-in-law, Mr. M. R. Mitchell, until his death on December 26th, 1920. His body was taken to Tyler, Tex., and buried beside that of his wife.

In 1882, Mr. Lynch was married, in Galveston, Tex., to Mrs. J. C. Kelly (whose maiden name was Virginia K. Mitchell). Mrs. Lynch died in 1909. There were no children, and Mr. Lynch is survived by a sister, Miss Nora Lynch, who resides in Cork, Ireland.

Mr. Lynch was noted as a Locating Engineer, and many railroads, especially of those in Texas, show evidence of his skill as such, as well as his partiality for tangents. In addition to his railroad work, he was a good "all round" engineer, having received his early technical training before engineering work became specialized and when all kinds came to one engineer.

With a pleasing appearance, equable temper, engaging manner, and an immense fund of dry humor, "Judge" Lynch, as he was usually called, was a true friend, and as he was a general favorite with all who knew him, he will be greatly missed.

Mr. Lynch was elected a Member of the American Society of Civil Engineers on May 4th, 1892.

CHARLES JOSEPH McDONOUGH, M. Am. Soc. C. E.*

DIED DECEMBER 11TH, 1920.

Charles Joseph McDonough, the eldest son of Charles J. and the late Mary (Hynes) McDonough, was born on December 31st, 1873, in Buffalo, N. Y. His parents were also born in Buffalo, his grandparents on both sides coming to the United States from Ireland when they were very young. His grandfather and father were well known contractors and builders, and the latter also occupied the position of Assistant Superintendent of Public Works, Western Division, of the New York State Canals during various Democratic administrations for periods aggregating twenty-four years, his last connection terminating on March 1st, 1921.

Charles Joseph McDonough received his early education in St. Bridget's Parochial School. He learned the carpenter trade during his vacations while attending St. Joseph's Collegiate Institute, Buffalo, from which he was graduated with the highest honors in almost every subject. From this school he entered Manhattan College, New York City, where he received the degree of Bachelor of Arts in 1892. He next attended the Rensselaer Polytechnic Institute, Troy, N. Y., from which he was graduated in the Class of 1897, with the degree of Civil Engineer.

His first professional work was with George A. Ricker, M. Am. Soc. C. E., in 1897-98, on the Buffalo Traction Company's (now International Railway Company), trolley line and on dock work for the Buffalo Union Furnace Company.

On June 15th, 1898, he entered the service of the City of Buffalo, N. Y., as Leveler, in the Department of Public Works and remained in this service for some time.

From 1900 to 1902, Mr. McDonough was associated with a firm of general building contractors in Syracuse, N. Y. He was appointed Assistant Engineer (temporary), in the Department of Public Works, Buffalo, N. Y., in May, 1902, but resigned on October 4th, 1904, to accept a position with the State of New York. While with the City of Buffalo, he was principally engaged in field work in connection with paving and, at the time of his resignation, was in charge of paving repairs covering a maintenance of upward of 3 000 000 sq. yd.

George H. Norton, M. Am. Soc. C. E., City Engineer of Buffalo, in speaking of Mr. McDonough's work, says, "for his age he displayed a marked technical and executive ability and we regarded him as a distinct asset to any engineering office. I think his work with the State of New York invited the same confidence."

After his appointment as an Assistant Engineer on the New York State Barge Canal in 1904, Mr. McDonough was assigned to the Champlain Canal Section, at Whitehall, N. Y., under Mr. John R. Kaley, but soon after was sent to the Barge Canal Office at Albany, N. Y. In January, 1905, he was transferred to Residency No. 11, at Tonawanda, N. Y., under Thomas W. Barrally, M. Am. Soc. C. E., and placed in charge of field surveys for the appropriation of lands and was also employed at office work in the preparation of contract drawings, until the early part of 1907, when he was placed in charge of construction work on Contract No. 19 and served in this capacity during 1907 and 1908.

* Memoir prepared by Emile Low, M. Am. Soc. C. E.

On January 1st, 1909, Mr. McDonough was appointed Resident Engineer, succeeding Mr. Barrally, who was appointed Division Engineer of the Western Division. The work carried on under the supervision of the Resident Engineer was: Residency No. 10-*B*, extending 100 ft. east of the Gasport Bridge, westerly to the Sulphur Springs guard lock, 11.7 miles, including Contracts Nos. 66, 67, and 40; Residency No. 11, extending from the Sulphur Springs guard lock at Pendleton to and through the City of Buffalo, the principal contract, No. 19, extending from the Sulphur Springs guard lock to the Delaware Street Bridge, Tonawanda, 12.46 miles. These four contracts extending over a distance of 24 miles involved an expenditure of about $6 000 000, and included the enlargement of the canal in solid rock through the City of Lockport and the construction of Locks Nos. 34 and 35, as well as the long solid rock cut west of Lockport.

Mr. Barrally, now City Engineer of Tonawanda, N. Y., with whom Mr. McDonough served for a number of years, writes of him as follows:

"In a work of such magnitude as the Barge Canal, each individual must of necessity be judged by his connection with the work as a whole and in this respect it will be seen from the record that he contributed in a large measure toward the consummation of this great enterprise. He was capable, energetic, and intelligent. With courtesy and patience, he directed his organizations and was rewarded by love and respect. His integrity was so obvious that it seemed to follow from a law of nature rather than choice.

"In private intercourse, he possessed a great fascination; this arose out of his ability to clothe all his thoughts in graphic and picturesque forms, which set him apart as an original character. He had, in a large measure, an inheritance of all those delightful traits so generously bestowed on the race from which he was descended. These are imagination, generosity, impulsiveness, and great originality in form of expression.

"In making a definite portrait of the character of a man we take the testimony of all who knew him and thus form a composite picture. From all this testimony I have reached the conclusion that while all will express an admiration of his ability as an Engineer, the picture of the Man will always remain most distinct and clear in the memory of all his friends and associates.

"Bishop Brent has recently stated that 'the master key of life is to be found in friendship,' and so we believe he found the simple principles on which he based his own life and left it to us as an example to follow."

This statement is supplemented by Frank M. Williams, M. Am. Soc. C. E., State Engineer of New York, who says: "From my personal acquaintance with Mr. McDonough and the opinions of other people associated with him, I heartily concur in the appreciation of him as stated by Mr. Barrally."

In January, 1912, Mr. McDonough was appointed Resident Engineer in charge of all Barge Canal terminal work, both design and construction, in Buffalo, and continued in charge of this work until November 1st, 1913, when he was appointed a Division Engineer in the New York State Highway Department from the first list of eligible Division Engineers established by the State Civil Service Commission after an open competitive examination. He was in charge of the Syracuse Division until 1915 when he was transferred to the Buffalo Division, where he was located at the time of his death, on December 11th, 1920. He was seated at his desk in his office and dropped dead following a sudden attack of apoplexy.

His wife received the following message from Col. Frederick Stuart Greene, M. Am. Soc. C. E., late Commissioner of Highways, State of New York:

"I am deeply grieved at the sorrow which has befallen you. The State of New York will share with you the loss, for your husband was not only a good engineer, but a man who set a high standard for others in the Department to follow."

In the death of Mr. McDonough, Buffalo lost an estimable citizen, a man of worth and character. Called at the climax of his mature manhood, he is mourned by a wide circle of men and women who knew of his sterling qualities. Indeed, Mr. McDonough's splendid capabilities as an engineer, his dependable judgment in large affairs, his scrupulous integrity, his fair mind, and his fine temper, were better recognized afar than in the limited area of his home city. It is to his skill and ceaseless vigilance that the excellent condition of the highways of Western New York is due.

On June 8th, 1910, Mr. McDonough was married to Miss Ella Molloy. He is survived by his widow, a son, Charles J., Jr., age 9 years, a daughter, Helen, age 8 years, and also by his father, three brothers, and four sisters.

He was at one time a member of the Knights of Columbus, Buffalo Council No. 184, but resigned on December 31st, 1912.

Mr. McDonough was elected a Member of the American Society of Civil Engineers on December 5th, 1911.

WALLACE McGRATH, M. Am. Soc. C. E.*

DIED DECEMBER 5TH, 1909.

Wallace McGrath, the son of the Rev. Thomas McGrath, was born on March 11th, 1844, in Wellsville, Ohio.

During the Civil War, he entered the Union Army as a Private, and was mustered out at the close of hostilities with the rank of Captain. Shortly thereafter he went to Kansas and took charge of the construction on the eastern end of the Kansas Pacific Railway, now a part of the Union Pacific System between Kansas City, and Denver, Colo. Later, after having been engaged in commercial pursuits in Topeka, Kans., for several years, he returned to Ohio and became Chief Engineer of the Scioto Valley Railway, extending from Columbus, Ohio, to the Ohio River and now a part of the Norfolk and Western Railway System. In 1880, he built the extension of that line from Portsmouth to Ironton, Ohio.

In January, 1881, Captain McGrath took charge of the surveys for the New York, Chicago and St. Louis Railway (the "Nickel Plate") between Cleveland, Ohio, and Buffalo, N. Y. He suggested the steel trestles subsequently used to span the numerous gorges crossed by this line between Cleveland and Buffalo, which for a time had caused the promoters to doubt whether the enterprise could be completed successfully. He became Superintendent of Construction and, later, personal Consulting Engineer for Mr. Calvin Brice and Gen. Samuel Thomas who were interested in the syndicate that built the "Nickel Plate Road," and who thereafter became well known promoters of railroad enterprises.

Captain McGrath was a man of magnificent physique, great personal magnetism, and of penetrating and sagacious intellect. He was a born leader of men. He hated pretense and sham, and refused positions which in his view involved catering and servility. He had the faculty of detecting a man's special value for a particular work even when obscured by less desirable surface characteristics. His rise in railroad engineering covered a relatively short period interrupted by the years of his mercantile experience and the years of depression following the panic of 1873, and yet he had become so widely and favorably known among men of financial standing in the railroad world that only the entire loss of health prevented him from becoming one of the outstanding railroad men of his time, a man similar to the late Albert Fink, F. Am. Soc. C. E., or the late John Newell, M. Am. Soc. C. E.

Friends who knew him during his long invalidism, the result of the exposure incident to his military service, were amazed and inspired by his indomitable courage. Although confined for nearly twenty-five years within constantly narrowing limits Captain McGrath never whimpered and always had a pleasant word for his friends and acquaintances.

Although prevented by his condition from doing any work involving responsibility or worry, he retained his mental powers to the last. He occupied the years of his confinement in wide reading, in the acquisition of the French language, and in keeping abreast with the advance in engineering knowledge, especially through

* Memoir prepared by E. H. Lee, M. Am. Soc. C. E.

the various publications of the Society, which were a great source of pleasure and interest to him.

Captain McGrath was twice married. He lost his first wife in 1874. He married his second wife, Miss Sybil Fairfield, of Columbus, Ohio, in 1878, and his death left her a widow with three daughters.

Captain McGrath was elected a Member of the American Society of Civil Engineers on September 5th, 1883.

ANICETO GARCIA MENOCAL, M. Am. Soc. C. E.*

DIED JULY 21ST, 1908.

Aniceto Garcia Menocal, the son of Gabriel Menocal, a Cuban planter, was born in Havana, Cuba, on September 1st, 1836. Educated in the schools of Havana, he came to the United States in 1858, and entered Rensselaer Polytechnic Institute, from which he was graduated with the Class of 1862.

Mr. Menocal then returned to Cuba and was connected with the "Vento" Water Works, supplying the City of Havana, as an Assistant Engineer from 1863 to 1865, and as Chief Engineer in charge of location and construction from 1865 to 1869. This enterprise involved many new and important features and, at the time, was considered to be an engineering problem of great magnitude. Since its completion it has required no modifications, sufficient foresight having rendered it capable of supplying present and future needs for many years to come.

Mr. Menocal returned to the United States in 1870, and for two years was connected with the Department of Public Works of New York City. In 1872, the Federal Government having decided on the need of an interoceanic canal to handle the rapidly growing commerce of the United States, President Grant requested Gen. George B. McClellan, then in charge of the Department of Docks of New York City, to suggest the name of an engineer to act as Chief of the Surveying Expedition, and Gen. McClellan strongly recommended Mr. Menocal. This expedition was engaged on a survey of the Nicaragua route from 1872 to 1874, the results of which were published as Executive Document No. 57 and presented to the Forty-third Congress.

Further investigations being desired, Mr. Menocal was appointed by the Federal Government as Chief Engineer of the Panama Canal Surveying Expedition in 1874–75, which was also reported on in a voluminous executive document. The results of these surveys demonstrated to Mr. Menocal the impracticability of a sea-level canal at Panama, and he favored the construction of a lock canal at Nicaragua as affording a shorter route, engineering works of less magnitude, and much lower cost than even a lock canal at Panama. The question of cost was verified by subsequent events.

In 1874, Mr. Menocal was appointed a member of the Corps of Civil Engineers, United States Navy, and stationed at the Navy Yard at Washington, D. C.

Public interest in a maritime canal was shown at every session of Congress, and on April 19th, 1879, President Hayes delegated Mr. Menocal and Rear-Admiral Daniel Ammen, U. S. N., "as delegates to attend the International Conference under the auspices of the Geographical Society of Paris, for the purpose of considering the various projects across the American Isthmus. * * * the importance and magnitude of the projected enterprise being such as to command attention, especially on the part of those countries whose trade is to be affected in a marked degree by the success or failure of the scheme." Mr. Menocal was "to take part in the discussions of the conference and communicate such scientific, geographical, mathematical, or other information as he possessed, that was desired or deemed important." The plans adopted at the Conference were against protests of eminent

* Memoir compiled from information on file at the Headquarters of the Society.

engineers familiar with the route selected, and the American delegates made a voluminous report of the meeting to Secretary of State Evarts.

At the end of the Congress, President Grevy, of France, made Mr. Menocal a Chevalier of the Legion of Honor.

After his return from France, Mr. Menocal became associated with Gen. Grant, Levi P. Morton, Gen. McClellan, Rear-Admiral Ammen, and other prominent men of the time, in the organization of the "Provisional Interoceanic Canal Society", and in May, 1880, he secured for the Society a concession from the Republic of Nicaragua for the construction of a canal. Owing, however, to lack of support for its project, the Provisional Society was merged into a corporation known as the Maritime Canal Company of Nicaragua, with Mr. Menocal as Chief Engineer. The failure of the firm of Grant and Ward in May, 1884, however, overturned all the projected plans, and the concession held by the Company was allowed to lapse.

In 1881, while Consulting Engineer of the Bureau of Yards and Docks, U. S. Navy Department, and a resident at the Washington Navy Yard, Mr. Menocal formulated the plans for the Government's large and only gun factory and by great foresight designed a plant which with only small modifications can turn out in quantities the huge naval guns of the present day. In this work he was assisted by Civil Engineer Robert E. Peary, U. S. N., the discoverer of the North Pole, who had worked with Mr. Menocal for a number of years after his entry into the Navy. In connection with the construction of this gun factory, Mr. Menocal presented a paper entitled "Sub-Aqueous Underpinning",* before the Annual Convention of the Society on May 17th, 1882.

From 1876 to 1884, Mr. Menocal had made various surveys in Nicaragua other than those for the interoceanic canal projects, among which were extensive surveys for improving the harbor of Greytown and the navigation of the Rio San Juan, and for a canal connection between Lakes Managua and Nicaragua known as the "Tipitapa Route", all of which were fostered by the Nicaraguan Government and described in a paper entitled "Inter-Oceanic Canal Projects",† presented before the Society on November 19th, 1879.

In 1886, the American Geographical Society took up the discussion of the subject of an interoceanic canal, and the matter of organization and incorporation of companies to accomplish this object was again considered by an association of prominent men. In the meantime, Mr. Menocal was engaged in an exhaustive study of the progress being made on the Panama Canal, as well as of the mechanical plant used there, and made a lengthy report on the results of his investigations to those interested in maritime canal affairs.

Under an Act of Congress approved February 20th, 1887, the Maritime Canal Company of Nicaragua was organized with Mr. Menocal as Chief Engineer. He obtained a new concession from the Nicaraguan Government dated April 24th, 1887, and another from the Government of Costa Rica which was ratified on August 9th, 1888.

The engineering forces left the United States for Nicaragua in November, 1887, to begin the work of locating the canal and, by 1890, the Maritime Company had finished the location surveys and undertaken much of the preparatory con-

* *Transactions*, Am. Soc. C. E., Vol. XI (1882), p. 181.

† *Transactions*, Am. Soc. C. E., Vol. VIII (1879), p. 311.

struction work. Seven large ladder dredges were purchased from the contractors of the practically defunct Panama Canal and, by 1892, buildings for the accommodation of the forces and the plant had been erected at Greytown, jetties had been built, a channel 500 ft. wide and 15 ft. deep to the previously inaccessible sea, had been opened for navigation, 11 miles of standard gauge track laid, piers erected, 60 miles of telegraph lines constructed, 2 miles of canal excavated and 11 miles deforestated, etc., when all work had to be stopped due to lack of funds caused by the financial panics in the United States and Europe.

A bill introduced into the Senate, on January 10th, 1891, favoring Government control and option on the assets of the Canal Company failed, as well as other measures advocated in 1895 by President Cleveland, and in 1897 by President McKinley. In the meantime, the concessions of the Maritime Canal Company had expired, and further action was delayed by the war with Spain.

From 1872 to 1897, Mr. Menocal had devoted his energies to the furtherance of American trade through the construction of an interoceanic canal, in which pursuit he had been opposed by all manner of interests, political and commercial. During his connection with the Maritime Canal Company, he made two trips to Europe hoping there to obtain funds for the completion of the Nicaragua Canal, but although he awakened interest in the project in England and Belgium, the financial world was in such straits that help was not obtainable from either country. During this time Mr. Menocal was constantly associated with the prominent men of the world, rulers, statesmen, scientists, financiers, etc.

Failing health and advancing years had now begun to curtail his professional work, and, in 1898, he retired from the Navy with the rank of Commander. From 1892 to his retirement, he had been detailed as Civil Engineer first at the Navy Yard at Norfolk, Va., and afterward to the Brooklyn Navy Yard.

At the conclusion of the war with Spain, he was ordered to Havana, Cuba, as one of the Naval Commissioners to take over the Spanish naval properties at that place. On the completion of this work, he was sent, in 1900, as a member of another Board to establish a Naval Station at Olongapo, Philippine Islands, and after a lengthy sojourn there on board the U. S. S. *Brooklyn,* he returned to the United States.

In 1902, the need of a Naval Coaling Station on the African Coast was noted, and Mr. Menocal was sent to make investigations and to locate, if desirable, a station in Liberia, the party being conveyed there on the U. S. S. *San Francisco.*

In 1906, he was employed by the Cuban Government as Chief Engineer to investigate the control of floods in the Roque Valley and the irrigation projects of the Northern Provinces of Cuba.

In addition to the papers and reports mentioned, Mr. Menocal had written many others, among which were "Channel Improvements, Washington Navy Yard", describing the channel improvements made in the Potomac River necessitated by the rapid growth of the Navy, which was published in the *Proceedings* of the U. S. Naval Institute; a "Report of the United States Surveying Party, 1885" describing interoceanic canal surveys; "Nicaragua Canal Construction Company Report, 1890"; a paper read at the Fourth International Congress on Inland Waterways, held at Manchester, England, in July, 1890, entitled "The Nicaragua Canal, Its

Design, Final Location, etc."; a paper read at the World's Columbian Water Commerce Congress, held at Chicago, Ill., in 1893, entitled "Nicaragua Canal", and a paper on "The Panama Canal*", presented before the Society in February, 1906. Abstracts from Mr. Menocal's papers and reports were reprinted in various engineering papers, such as the *American Engineer, The Railroad Gazette,* and the *Engineering News,* and several authors also embodied the results of his efforts in book form, namely, Gen. Grant's "The Key to the Pacific", Archibald Colquhon's "Uncle Sam's New Waterway", William Simons, Rear-Admiral Ammen, and many others.

Mr. Menocal died at his home in New York City, on July 21st, 1908, after a long illness from arterio-sclerosis. He was buried in Arlington Cemetery, Washington, D. C.

In 1866, he was married to Miss Elvira Martin, of Cuba, who, with three sons, survived him.

He was a member of various engineering societies and scientific societies in the United States and Europe as well as of several commercial bodies, namely, Engineers Club of Philadelphia, Rensselaer Society of Engineers, Geographical Society of Antwerp, American Association for the Advancement of Science, United States Naval Institute, National Geographic Society, San Francisco Chamber of Commerce, etc.

Mr. Menocal was elected a Member of the American Society of Civil Engineers on February 3d, 1875.

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* *Transactions*, Am. Soc. C. E., Vol. LVI (1906), p. 197.

WILLIAM HARLEY MOORE, M. Am. Soc. C. E.*

DIED SEPTEMBER 5TH, 1920.

William Harley Moore was born on September 12th, 1860, in Limerick, Ireland. He was the son of William Harley Moore and Mary Elizabeth Ledger Moore, and the grandson of the late Zackariah M. Ledger, all of Limerick, Ireland. His uncle, James C. Ledger, was a prominent engineer of irrigation and bridges in India, and built the Great Bridge over the Narbudda River. Mr. Moore studied at Queen's College, Cork, where he was awarded a number of first prizes in Mathematics and a prize in Philosophy. In three years he had won his B. A. degree with first scholarship in Engineering, and in the next two years was awarded the degree of Bachelor of Engineering. He took his degree of Master of Engineering at the Royal University of Dublin, and won first class Exhibition and first class Honors. Queen's College later conferred on him the honorary degree of Master of Arts.

On completing his University work, Mr. Moore came to America, and in 1885 and 1886 was employed as a Draftsman in the Bridge Engineering Department of the New York Central and Hudson River Railroad. In May, 1886, he entered the employ of the New York, New Haven and Hartford Railroad, with which Company he continued until his death. He was appointed Bridge Engineer of the New Haven Railroad in January, 1889, and Engineer of Structures in May, 1913.

Nearly all the four-track bridges between New York City and New Haven, Conn., were designed and built under his supervision. Many six-track bridges on the Harlem River Branch, between New York City and New Rochelle, N. Y., including several large draw-bridges, were also built by Mr. Moore. He also either rebuilt or strengthened many of very large bridges of the New York, New Haven and Hartford System, among which were the large bridges over the Connecticut River, at Warehouse Point, Middletown, and Lyme, Conn., and the bridge over the Cape Cod Canal at Buzzard's Bay, Mass. The Thames River Bridge, at New London, Conn., was the last bridge designed and built under his supervision, and will remain as a monument to his ability.

Mr. Moore was a member of the American Railway Engineering Association, which he joined in 1900, and served on many of its important committees on iron and steel structures. He was made Chairman of the Sub-Committee on Column Tests, Committee XV, in June, 1912, and served in that capacity until the time of his death. He was also a member of the American Society for Testing Materials and the American Railway Bridge and Building Association.

Mr. Moore was not married. He leaves as his nearest relatives, his mother, Mrs. Mary Elizabeth Moore, and a sister, Mrs. George B. Chamberlin, both of whom reside in Brookline, Mass., and several nephews and nieces. He died suddenly on September 5th, 1920, at his residence in New Haven, Conn., of heart failure, resulting from bronchial trouble to which he had been subject for many years.

Mr. Moore's great technical ability was recognized by all who knew him professionally, and his loss will be severely felt in his various fields of professional

* Memoir prepared by Clarence Blakeslee, M. Am. Soc. C. E.

activity. His artistic tastes led him into a wide study of music and literature from which he gained keen pleasure. His taste for the æsthetic made him a most valuable and congenial companion at all gatherings where music or the drama contributed to the entertainment.

He was a good friend, a man among men, actuated by the highest ideals, and sympathetic in all his associations with his fellows. He was a genial and dignified gentleman, filling a large place in engineering circles, where his loss has caused deep sorrow.

Mr. Moore was elected a Member of the American Society of Civil Engineers on June 4th, 1895.

EMORY WASHBURN MUENSCHER, M. Am. Soc. C. E.*

DIED APRIL 15TH, 1917.

Emory Washburn Muenscher, the son of the Rev. Joseph Muenscher and Ruth Washburn Muenscher, was born at Fall River, Mass., on February 6th, 1834. There is a tradition that his great-grandfather, John Muenscher, came to America with the Hessians during the Revolutionary War as a Bandmaster. His maternal ancestors were of Puritan stock, their line tracing back to the *Mayflower* colonists, and a number of them served with distinction in the Colonial Army during the Revolution.

Mr. Muenscher was graduated in 1850 from the Classical Department of Kenyon College, at Gambier, Ohio. In 1854, he entered the Scientific Department of Union College, Schenectady, N. Y., from which he was graduated in 1856.

From October, 1851, to April 1853, he was employed as Rodman with the Springfield, Mt. Vernon, and Pittsburg Railroad Company, and from April to October 1853, with the Steubenville and Indiana Railroad Company as Compassman.

After his graduation from Union College, Mr. Muenscher again entered the employ of the Springfield, Mt. Vernon and Pittsburg Railroad Company as Assistant Engineer, but gave up his position to take up the profession of teaching in which he was engaged for the next five years.

On the first call for volunteers by President Lincoln, subsequent to the attack on Fort Sumpter, Mr. Muenscher enlisted, on April 20th, 1861, in the Union Army and was made Sergeant of Company B, 4th Ohio Infantry. Before his enlistment expired he was commissioned as First Lieutenant and appointed to the 30th Ohio Volunteers. At this time he re-enlisted for three years, and was promoted to the rank of Captain in August, 1862, and to that of Lieutenant-Colonel in January, 1865.

Colonel Muenscher served with distinction in the campaigns of West Virginia under Generals Rosecrans and Cox, in Virginia under Gen. McClellan, and, later, during the siege of Vicksburg. He had the honor of commanding the troops which opened the Battle of Missionary Ridge, and was in command of his regiment during Sherman's March to the Sea. His regiment was a part of the fifteen Army Corps once commanded by Gen. Logan. He took part in the battles of Antietam, South Mountain, the second battle of Bull Run, Vicksburg, Missionary Ridge, Dallas, Kenesaw Mountain, Atlanta, Ezra Church, Jonesboro, and Fort McAllister, going through them all and having been wounded only once, at Antietam.

At the close of the Civil War, Colonel Muenscher devoted himself to his profession with gratifying results. He served from March to December, 1865, as Assistant Engineer on the construction of the Marietta and Cincinnati Railroad, and from January, 1866, to June, 1867, as Chief Engineer on the surveys for the Eel River Railroad, the Goshen and Warsaw Railroad, and the Warsaw and Peru Railroad in Indiana.

Following this engagement Colonel Muenscher was appointed as Assistant Engineer on the Grand Rapids and Indiana Railroad, with headquarters at Grand Rapids, Mich., where he remained until June, 1869, when he was made City Engineer of Grand Rapids, having been the first City Engineer appointed by its Board

* Memoir prepared by Edward D. Rich, M. Am. Soc. C. E.

of Public Works. He served in that capacity until May, 1877, with the exception of one year, 1871-72, when he was in charge of surveys for the Grand Rapids Water-Works, the Grand Rapids and Holland Railroad, and the Grand Rapids and Lansing Railroad. During his incumbency as City Engineer, Colonel Muenscher had charge of the construction of the city water-works and designed and partly built the combined system of sewerage for the municipality. As there were few well sewered cities in the United States at that time, and very little literature on the subject, Colonel Muenscher did pioneer work along these lines. During the years which have elapsed since this work was constructed, little of it has proven to be inadequate, nearly all of it still being in service.

From May, 1877, to March, 1881, Colonel Muenscher was engaged in private practice as a Consulting Engineer, and from March, 1881, to June, 1884, he served as Chief Engineer on the construction of various railroads among which were the Belleville and Carondelet Railroad, the Kaskaskia, St. Elmo, and Southern Railroad, the Vandalia and Springfield Railroad, the Lexington and Utica Railroad, etc.

In June, 1884, he returned again to the private practice of his profession until January, 1885, when he was elected County Surveyor of Kent County, Michigan. He resigned that position in May, 1885, to become City Engineer of Manistee, Mich., which office, together with that of County Surveyor and County Road Engineer of Manistee County, he held at the time of his death. He had also served during this time as Chief Engineer of the Chicago, St. Louis and Paducah Railroad from March, 1887, to December, 1888, and of the Manistee and Grand Rapids Railroad from September, 1889, to 1906.

He was stricken with apoplexy while engaged in mailing personal invitations to friends to attend a patriotic meeting for the purpose of encouraging enlistments for the World War. His death occurred at his home in Manistee, Mich., on April 15th, 1917.

Colonel Muenscher was well known in State and National Engineering Societies. He became a member of the Michigan Engineering Society very soon after its organization and served as its President in 1893 and 1894, and was made an Honorary Member in 1915. He was a Master Mason of high standing and a devoted member of the G. A. R., having served as Commander of the James F. McGinley Post in 1893 and 1895, and from 1903 until his death.

Colonel Muenscher was a gentleman of the old school whom it was a pleasure to meet and to know. He had that fine sense of courtesy, consideration, and helpfulness which is such an inspiration to the younger members of the Profession, and possessed it in such a degree as to mark him as a rare spirit among his fellows. During his active years he was an original worker of no mean ability, and his practice was always characterized by painstaking care, foresight, and strict integrity. In his community he was an honored and respected citizen whose memory stands as an example of a well spent life devoted to the welfare of his country, his fellow men, and his family.

After the surrender of Vicksburg, Colonel Muenscher obtained a leave of absence and was married on August 22d, 1863, at Gambier, Ohio, to Miss Sarah E. Johnson of New York City. He is survived by his widow and two daughters, the Misses Josephine and Lulah Ruth Muenscher.

Colonel Muenscher was elected a Member of the American Society of Civil Engineers on July 5th, 1893.

ORLANDO WHITNEY NORCROSS, M. Am. Soc. C. E.*

DIED FEBRUARY 27TH, 1920.

Orlando Whitney Norcross was born in Clinton, Me., on October 25th, 1839. He was a typical example of the self-made man. When eleven years of age, his father's death placed on him the duty of contributing a share to the family support, and he then began a life of activity and industry that never flagged until the hour of his death.

In his chosen trade of carpenter, he easily became the Master Builder. Following his military service (1861-64) as a member of the First Massachusetts Heavy Artillery, he and an elder brother organized the firm of Norcross Brothers which operated as Contractors and Builders in the vicinity of Salem and Swampscott, Mass. In 1868, offices were opened by the firm in Worcester, Mass., and thereby was begun and developed the great building and contracting business which became national, and international, in its scope.

The name "Norcross" was known wherever a building of any prominence or distinction was contemplated, and cannot be forgotten in any place where a structure erected by the organization still stands. A list of the buildings, monuments, memorials, and other construction work erected by Norcross Brothers, or the Norcross Brothers Company (the business having been incorporated soon after 1897, when the older brother retired), would trespass heavily on available space, and a selection of the notable ones would include a very large part of the entire list. Those in which Mr. Norcross took the most pride were the Allegheny County Court House, Pittsburgh, Pa.; Trinity Church, Boston, Mass.; Rhode Island State House, Providence, R. I.; New York Public Library; the remodeling of the White House during the term of the late President Roosevelt; the removal of Henderson's Point, Kittery Navy Yard, and educational buildings for the great colleges and universities—Harvard, Yale, Princeton, Brown, Columbia, and others.

Mr. Norcross was one of three Commissioners appointed by the Secretary of the Treasury, in 1875, to examine and report on the condition of the Chicago Federal Building. A later commission modified many of the conclusions of the first commission, but eventually the findings of the Norcross Commission were verified and justified.

In addition to actual construction operations, Mr. Norcross aided materially in the development of industries in lines closely related to building work. The development and expansion of the quarries in Milford, Mass., produced the beautiful Milford pink granite, the most popular of all American granites. He also developed the sandstone quarries of East Longmeadow, Mass. The Milford granite and the Longmeadow sandstone form a combination that has never lost its popularity. The granite properties at Stony Creek, Conn., producing the handsome Branford granite, were developed by him. This is the stone demanded by architects and sculptors for monumental and memorial purposes, especially in combination with bronze statues and tablets. Through his perseverance immense marble-producing quarries and works were developed in Manchester and Dorset, Vt., and Pickens County, Georgia. In connection with the work of the organization there grew up great wood and iron-working establishments in Worcester and the

* Memoir prepared from information furnished by Edward F. Miner, M. Am. Soc. C. E.

heavier structural steel business in Eastern Massachusetts. It was often truthfully said by Mr. Norcross that Norcross Brothers were first-hand producers of more lines of work entering into building construction than any other organization doing a general building business.

Enjoying the confidence of architects, engineers, and owners, a confidence well deserved, and never misplaced, there was no question but that work entrusted to him, and undertaken by him, would be conscientiously and well performed. During his business life there were many changes: New methods appeared and were improved; engineering and construction features were revolutionized; steel and concrete took their places in the construction field, and Mr. Norcross contributed largely to the practical use of these materials. To him is to be credited the flat-slab construction of the reinforced concrete floor; it was his invention. The first tests and practical use to which it was put involved the use of ordinary poultry netting for the reinforcement, and from that very primitive type there grew the present forms and types of reinforced concrete in many of its uses. The decision of the Court, ending the controversy as to priority, is stated in the following words:

"The review of the prior art, the long and fruitless endeavor before the disclosure of this patent to find a way to construct concrete floors for large buildings without beams or girders, the gradual approach to the desideratum, its first conception and reduction to practice by Norcross, and its large and successful subsequent use, have satisfied that this flooring and his method of construction were not within the reach of the mechanic skilled in the art, but were novel and useful and the product of the genius of the inventor."

Mr. Norcross was never friendly to the "skyscraper". His terse remark concerning that type of structure was: "In a country where land is as plentiful and as low in cost as in the United States, there is no valid excuse for its existence."

The word "genius" was often applied to Mr. Norcross. He invariably objected and stated that what was mistaken for the reward of genius was really the compensation for hard work. Always active, always busying himself with some task, he was frequently advised to take a vacation and rest. His reply was "that rest was for the dead, and that he would have plenty of time for rest in the grave."

No problem of a practical nature was too difficult for his attention. The fact that grinding labor, painstaking effort, enormous energy and time, were involved in a problem never caused him to hesitate in finding the solution. The greater the obstacles and difficulties, the more resolutely he devoted himself to them.

Mr. Norcross was a liberal contributor to the church, to helpful institutions, and to practical charities; but his conception of extending help and charity was to guide those asking or needing it along the path of self-help. Realizing that all things attainable are had only through hard work, he, the persistent and tireless worker, believed that anything worth while was worth effort.

In the days of his early business career a neighborhood church society was organized, to which he and Mrs. Norcross gave their earnest support; and it was due largely to his great ability and practical knowledge that there was erected a modest, attractive, church structure, where St. Mark's Parish has been a harmonious and prosperous church family.

Mistakes he rarely dwelt on after having exposed the fault. His handling of transgressors was characteristic of the man. Always tolerant, forgiving, and hopeful of reform, he gave many men reason to be thankful to him for great

patience and leniency, for not until it was clear that improvement was apparently impossible would he abandon his effort for reformation.

Mr. Norcross was a member of the Boston Society of Civil Engineers, Boston Chamber of Commerce, Worcester Chamber of Commerce, Master Builders Associations of Boston and Worcester, National Geographic Society, American Society for the Advancement of Science, and local social clubs of Worcester. He was a Charter Member of the Worcester Builders Exchange, Director of the State Mutual Life Assurance Company, Trustee of Clark University, and a Vestryman of St. Mark's Protestant Episcopal Church.

Mr. Norcross was married in 1870, to Phoebe Ellen Sibley, of Salem, Mass. Mrs. Norcross died in 1905. Their family consisted of two sons and three daughters. Both sons died in infancy. On the 81st anniversary of his birth there was dedicated to him, in the Worcester City Hall, a memorial given by citizens of Worcester. This memorial is in the form of a bronze tablet, 6 ft. high by 4 ft. wide, backed by a panel of his Stony Creek granite. In addition to a life-size portrait in a bas-relief profile, the tablet bears this inscription:

"ORLANDO WHITNEY NORCROSS

"Born in Clinton, Me., Oct. 25, 1839
"Died in Worcester, Mass., Feb. 27, 1920

"Of sturdy Anglo-Saxon stock.
"Soldier of the Civil War.
"Exemplar of the Dignity of Labor.
"Friend of Learning.
"Zealous for the Public Welfare.
"Honored Citizen of Worcester.
"Master Builder."

Mr. Norcross was elected a Member of the American Society of Civil Engineers on October 31st, 1911.

ARSENE PERRILLIAT, M. Am. Soc. C. E.*

DIED OCTOBER 23D, 1920.

Arsene Perrilliat was born in Paris, France, on July 10th, 1865, while his parents were residing there temporarily, and was descended from a long line of prominent French ancestors. He was brought to New Orleans, La., by his parents, when only eighteen months old, and his early schooling was obtained there in well known private institutions and with prominent private instructors of the day. He afterward attended the University of Louisiana, which, in 1884, became Tulane University, and from which he was graduated with the degree of B. S., in 1883. In his class were numbered many students of recognized reputation and distinction in after life.

After his graduation, Mr. Perrilliat was employed at first on surveys for the Mississippi River Commission, and, later, became Assistant Chief Engineer for the New Orleans Cotton Centennial Exposition of 1884. From 1885 to 1888, inclusive, he taught at Tulane University with the rank of Assistant Professor of Physics, Mathematics, and Chemistry. During vacations, he took courses in higher mathematics at the University of Virginia, and worked in Edison's Laboratory in Schenectady, N. Y., specializing in electrical work.

In 1888, Mr. Perrilliat accepted the position of General Superintendent of the Edison Electric Light Company of New Orleans, and in the Fall of the same year was appointed, by Governor Francis T. Nicholls, a member of the Board of State Engineers of Louisiana, serving in that capacity until 1908. In June, 1897, the degree of Civil Engineer was conferred on him by Tulane University.

In 1900, with the late E. L. Corthell, Past-President, Am. Soc. C. E., of New York City, and J. A. Ockerson, Past-President, Am. Soc. C. E., of St. Louis, Mo., he was selected by the Secretary of War to represent the United States Government at the International Congress of Navigation, held that year in Paris, France, and was again so selected for a similar function at St. Petersburg, Russia.

Among the many duties imposed on him during his connection with the Board of State Engineers, was that of protecting the alluvial lands of Louisiana from overflow, with particular reference to those situated in the Atchafalaya Basin District, in which he took most pride, lifting the District from annual devastation from floods to a degree of security therefrom, justifying its classification as one of the most secure and richest portions of the State.

In 1908, Mr. Perrilliat severed his connection with the service of the State to become Managing Engineer of the Hercules Company, engaged in general engineering and contracting work. He, several times, went to Holland, Southern France, and Northern Italy to study and investigate the principles and methods of dike and levee construction in practice in those countries, afterward writing a number of valuable papers on the subject. He took great pride in the fact that he was the originator of constructing, by mechanical means, embankments for protection against the overflows of the Mississippi River, which resulted in the saving of vast sums of money to the State of Louisiana and the United States Government.

* Memoir prepared by Frank M. Kerr, M. Am. Soc. C. E.

In 1919, Mr. Perrilliat was especially selected by the Board of Commissioners of the Port of New Orleans, to serve as its Chief Engineer, and was thus professionally engaged at the time of his death on October 23d, 1920. His duties as Chief Engineer of this Board, charged with the improvement, on the most extensive and comprehensive scale, of the port facilities of New Orleans, called for the possession and exercise of the widest range of professional knowledge and skill, combined with business acumen. In all of these he was recognized as eminently qualified.

Mr. Perrilliat was a man of keen intellect, a wide reader, and a brilliant conversationalist in several languages. In addition to his high professional attainments and standing, he was also closely identified with all the social and artistic activities of prominence, not only of New Orleans, but of other prominent centers of life in a number of other directions. He was as well a gentleman "par excellence", a prince of good fellows, kind hearted and charitable to a fault, and will ever be greatly missed by the many who had the privilege of knowing him.

He is survived by his widow, who before marriage was Charlotte Beatrice Hollywood, of Indianapolis, Ind., by a daughter, Mrs. George R. Fowler, Jr., a grandson, and a number of relatives, more or less distant.

Mr. Perrilliat was elected an Associate Member of the American Society of Civil Engineers on June 7th, 1893, and a Member on April 5th, 1899.

JOSEPH ALLEN POWERS, M. Am. Soc. C. E.*

DIED AUGUST 31ST, 1912.

Joseph Allen Powers, the son of Albert E. Powers, oil cloth manufacturer and then banker, of Lansingburg (now a part of Troy), N. Y., and of Lucy Clarke (Allen) Powers, was born on June 28th, 1858.

As a boy he showed an early and intense interest in things of a mechanical nature and an enthusiasm for fishing and hunting, but he was equally devoted to reading, burying himself in books, often of travel or adventure, for hours at a time. His taste in these three directions was maintained throughout his life.

Graduating in 1880 from Rensselaer Polytechnic Institute with the degree of C. E., Mr. Powers' first work was with the Bridge Department of the Delaware, Lackawanna and Western Railroad. In 1883, he accepted a position as Bridge Engineer of the Delaware and Hudson Railroad, which position he resigned in 1886, in order to take charge of an electric lighting plant for the City of Troy, in which his father was interested. Then followed a series of years devoted to the electrification of interurban car lines.

In 1888-89, he purchased the Glens Falls, Sandy Hill and Fort Edward horse-car line and was instrumental in its reorganization and reconstruction as the Hudson Valley Railway, now operating from Glens Falls to Waterford, N. Y., *via* Saratoga. At the time of its completion, this railway was said to be the longest interurban trolley line in the United States.

In 1890, Mr. Powers obtained control of the Burlington, Vt., horse-car railway running between that town and Winooski, and converted it into a trolley line—the first electric railway in the State. About this time, too, he built the St. Catherine's and Niagara Falls Electric Railway and converted the steam-operated road from Port Kent, N. Y., on Lake Champlain, to Keeseville, to a trolley road.

Mr. Powers was the promoter and then President of the Northern Adirondack Power Company, at Keeseville, N. Y., and President of the Powers and Mansfield Company, Contractors, of Troy.

On the death of his father in 1910, he was appointed sole administrator of his estate, and much of the remainder of his life was devoted to its settlement. He died on August 31st, 1912, and is survived by his wife, two sons and two daughters.

Mr. Powers' outstanding traits were: A faculty of intense concentration on the work in hand, an abundant and robust energy, combined with a spirited and genial manner, and a love of outdoor sports and good literature. As a man of affairs, he was forceful, and as an engineer he had marked ability which, in his chosen lines of mechanical and electrical work, was well recognized.

He was a member of the Engineers' Club of New York, the Rensselaer Society of Engineers, the Elks, the Island Golf, and the Pafraets Dael Clubs, an honorary member of the Troy Citizens' Corps, and a Trustee of All Souls Unitarian Church. For the last five or six years of his life Mr. Powers was a Director of the Santa Cecilia Sugar Company, located at Guantanamo, Cuba, where he spent several winters.

Mr. Powers was elected a Junior of the American Society of Civil Engineers on April 2d, 1884, and a Member on September 3d, 1890.

* Memoir prepared by Kenneth Allen, M. Am. Soc. C. E.

ISHAM RANDOLPH, M. Am. Soc. C. E.*

DIED AUGUST 2D, 1920.

The passing of Isham Randolph at his home in Chicago, Ill., has removed from the Profession an engineer of well known attainments, commendable public spirit, and charming personality.

Isham Randolph was born in Clarke County, Virginia, on March 25th, 1848, and was a descendant of William Randolph, of Yorkshire, England, who came to Virginia about 1674. His grandfather, Archibald Cary Randolph, first cousin to Thomas Jefferson, was a Captain of Infantry in the United States Army. Robert Carter Randolph, the father of Isham Randolph, who died in 1887 at the age of eighty, was a physician of New Market, Clarke County, Va. Mr. Randolph's mother was of the Nelson family of Virginia, which was actively identified with Virginian history during Revolutionary times. Thomas Nelson, born at Yorktown in 1716, was Secretary of the Colonial Council of Virginia, and was popularly known as "Secretary Nelson". His three sons became officers in the American Army under Gen. Washington.

Isham Randolph began his career without technical school training and acquired physical and mental equipment for the Engineering Profession by experience and the improvement of opportunities as they came to him. His professional record shows steady advancement from one position to the next higher, a progress which, in many instances, would fail to satisfy the ambition of the young engineer of to-day. Such training, however, acquainted him in a practical way with all the details, minor and otherwise, of professional demands, and became an important factor in the various phases of his career.

When only twenty years old, he entered the employ of the Winchester and Strausburg Railroad as Axeman, and after the location was finished, he continued in the service of this road as Rodman until its completion. In 1870, he was engaged as Levelman on the surveys for the Washington and Ohio Railroad, and, in 1871, served as Transitman on the surveys for the extension of the Lehigh Valley Railroad from Jugtown Mountain to Perth Amboy, N. J.

In March, 1872, Mr. Randolph became Transitman on the extension of the Baltimore and Ohio Railroad to Chicago, Ill. Later, as Resident Engineer for the company, he built 27 miles of the road and the roundhouse and shops at South Chicago. During five years of railroad work, he had risen from Axeman to Resident Engineer, acquiring by persistent effort the technical knowledge and training that was needed to make him proficient in each succeeding step of his work. It should be remembered, also, that field work for railroad surveys in those days was of the pioneer type which taxed the physical endurance and professional resourcefulness of the young engineer to the utmost.

Following his service with the Baltimore and Ohio Railroad, he became Assistant Engineer of the Scioto Valley Railroad, and, in 1880, he was appointed Chief Engineer of the Chicago and Western Indiana Belt Railway of Chicago, and rounded out his railroad experience in building terminals, freight houses, etc.

In 1885, after having been for seventeen years in the employ of railroad companies, Mr. Randolph opened an office in Chicago for general engineering work,

* Memoir prepared by C. G. Elliott, M. Am. Soc. C. E.

and, in the following year, was employed by the Illinois Central Railroad Company as Chief Engineer in the location and building of the Chicago, Madison, and Northern Railroad, and the Freeport and Dodgeville Line. This work having been completed, he resumed private practice in Chicago and, later, became Consulting Engineer for the Union Stock Yards, and for the Baltimore and Ohio Railroad. It should be noted that Mr. Randolph started his professional career twenty-two years before as Axeman on this railroad and had now been accorded the position of Consulting Engineer for the entire System.

One of the most notable works which was carried out under his direction was the Chicago Drainage Canal, for which he was appointed Chief Engineer on June 7th, 1893. Prior to the construction of the Panama Canal, this was the largest artificial canal ever built. The cost was $60 000 000 and fourteen years were required to construct it. From 1907 to 1912, Mr. Randolph was retained as Consulting Engineer of the Canal. At the conclusion of his services as Chief Engineer, the Board of Trustees of the Sanitary District of Chicago recorded its high appreciation of his work in terms not often accorded to members of the Engineering Profession, by adopting the following resolutions:

"Resolved: By the Board of Trustees of the Sanitary District of Chicago in regular meeting assembled this 24th day of July 1907, that we here record our high appreciation of the long and faithful services of Isham Randolph, as Chief Engineer of the Sanitary District, of his pre-eminent abilities and of the fine qualities of mind and heart which have endeared him to all who have had the good fortune to come into close personal relations with him. We regret the loss of the services of Isham Randolph as Chief Engineer and rejoice that, as Consulting Engineer, his skilled advice will still be available for the completion of the great work with which he has been so long and so closely identified; and be it further

"Resolved: That a minute of these resolutions be entered in the proceedings of the Board and that a copy, suitably engrossed, be prepared and presented to Mr. Randolph."

In recognition of his accomplishments in connection with this work, Mr. Randolph was awarded a gold medal by the Paris Exposition in 1900.

He was identified with the initiatory plans for the Panama Canal as one of the Board of Consulting Engineers appointed by President Roosevelt to recommend a plan for a canal. He was also one of the five members of the Board whose minority report was accepted by the President and Secretary of War, and on whose recommendations the Canal was finally constructed.

In 1908 he was again appointed by President Roosevelt as one of six engineers to accompany President-elect Taft to Panama to consider "whether or not there is any reason to change the plans upon which we are working?" This Board of Engineers made a report to the President on February 16th, 1909, unanimously approving the lock canal across the Isthmus.

Mr. Randolph designed and built, for the Queen Victoria Niagara Falls Park Commission, what is known as the "Obelisk" Dam across the Horseshoe Falls. It was built upright on the bank of the river, and was successfully tipped over across the stream, accomplishing the purpose for which it was designed.

As Chairman of the Internal Improvement Commission of Illinois, he assisted in the development of plans for a canal from Lockport to Utica, involving the design of five hydro-electric power plants for developing 140 000 h. p. The people

of the State voted $20 000 000 in bonds for the construction of this work. In connection with the further development of the State of Illinois, he was a member of the Illinois State Conservation Commission which was charged with the duty of examining and reporting on the natural resources of the State, and of the River and Lakes Commission which had jurisdiction over its rivers and lakes. He was also a member of the Chicago Harbor Commission which made a thorough study and an exhaustive report, together with plans, for a lake-front harbor in Chicago.

The City of Toronto, Ont., Canada, employed Mr. Randolph as Consulting Engineer on track elevation, and he was also made a member of the Water Supply Commission which made the plans for a new water supply and water-works for that City.

In recent years, he had been engaged in land reclamation work, serving as Consulting Engineer for the Little River Drainage District of Southeast Missouri and as Chairman of the Florida Everglades Engineering Commission which reported to the Trustees of the Internal Improvement Fund of the State of Florida on the reclamation of the Everglades. This report was published as U. S. Senate Document No. 379, in 1914.

Although denied the advantages of a college and technical education in his younger days, and the degrees which mark such attainments, Mr. Randolph's accomplishments were appreciated in later years by institutions of learning. On June 15th, 1910, the University of Illinois conferred on him the Honorary Degree of Doctor of Engineering; and, in February, 1913, the Franklin Institute of Philadelphia, Pa., conferred on him the Elliott Cresson Medal in recognition of "Distinguished Achievement in the Field of Civil Engineering".

Mr. Randolph was a Past-President of the Western Society of Engineers, a Member of the American Institute of Consulting Engineers, and a Fellow of the Royal Society of Arts of Great Britain.

The foregoing is the record of a long, active, and useful life spent in the Engineering Profession. It should be an inspiration to the engineering student of to-day, along the lines of persistent, conscientious, and diligent effort in the improvement of opportunities for the acquirement of technical knowledge and engineering experience.

Mr. Randolph had a broad vision of development possibilities in the projects which he directed. He was an active and earnest advocate of the measure now before the public to create a National Department of Public Works. As a writer of reports, his style was graceful and his expressions direct and clear. His tall figure was often seen at public gatherings where engineering topics and National improvements were under consideration. His affable and genial manners, in addition to his engineering attainments, gave him a wide acquaintance and influence with engineers and other public men throughout the United States, all of whom, it is safe to say, mourn sincerely because they shall see his face no more.

On June 15th, 1882, Mr. Randolph was married to Mary Henry Taylor, daughter of Capt. George Edmund Taylor, who, with three sons, Robert Isham Randolph, M. Am. Soc. C. E., the Rev. Oscar de Wolf Randolph and Spottiswood Wellford Randolph, survives him.

Mr. Randolph was elected a Member of the American Society of Civil Engineers on February 4th, 1903, and served as a Director in 1916.

AUGUSTUS VALENTINE SAPH, M. Am. Soc. C. E.*

DIED FEBRUARY 13TH, 1920.

Augustus Valentine Saph was born in San Jose, Cal., on October 26th, 1871. He was the son of Augustus Joseph and Naomi McPherson Saph, his father's occupation having been that of contractor and builder. His early education was received in the Grammar School and High School of his native town. He was graduated from the College of Civil Engineering, University of California, in 1894, returned the next year and received the M. S. degree, and acted as Instructor in Drawing and Mathematics from 1896 to 1900. He then continued his studies at Cornell University, graduating in 1902 with the degree of Ph. D. The following year Mr. Saph returned to the University of California as Instructor in Civil Engineering.

From 1903 to 1906, he was engaged as Structural Engineer, in charge of drafting, for the U. S. Reclamation Service at Reno, Nev., and then at Hazen, Nev., on the Truckee-Carson Project, having charge of the design of diversion dams and other structures. In 1906, he was employed by John B. Leonard, M. Am. Soc. C. E., Consulting Structural Engineer, on the design of bridges and buildings. From November, 1908, to May, 1911, he acted as Assistant Engineer in the office of the City Engineer of San Francisco, Cal., and designed the College Hill Viaduct, also Pumping Stations Nos. 1 and 2, the Ashbury Heights Tank, bridges, retaining walls, and four reinforced concrete chimneys.

From May, 1911, to August, 1912, Mr. Saph was Assistant State Engineer of California, acting as Chief Engineer for the Board of State Harbor Commissioners at San Francisco, in charge of design and water-front construction and maintenance. After leaving the Harbor Board, he became Engineer on Special Design of the Spring Valley Water Company, of San Francisco, in charge of the design of the water tower for the Calaveras Dam.

In 1914, he opened his own office in San Francisco as a Consulting Engineer, specializing in structural engineering.

Mr. Saph was a great man on detail, and was best pleased when working on intricate problems. He reminded the writer of the following lines by Prof. J. S. Blackie in "The Wise Men of Greece", describing our ancient brother Pythagoras:

> "Mathematics is a long aisle or porch
> Through which our sense-bound groping intellect
> Into the temple of pure reason passes,
> Where sun-bright gods are throned. 'Tis strange to see
> How he will sit, and look, and brood, and think
> His soul out of itself into a square,
> Or circle, or triangle. In my school
> A young athlete once found him with fixed gaze
> Bent o'er the dust, where he had traced a web
> Of inter-crossing figures; there he sat,
> Motionless as a sphynx in Memphian sands,
> Or Niobe on Sipylus; when, behold!
> Even as a flash from thunder-pregnant cloud,
> Sudden he starts, and like an uncaged bird
> That flaps free wings, and shrills with ecstasy.
> He cried 'Eureka—I have found it!'"

* Memoir prepared by John B. Leonard, M. Am. Soc. C. E., and Edward L. Soulé, Assoc. M. Am. Soc. C. E.

Mr. Saph passed away very suddenly during the influenza epidemic and was laid to rest by his brethren of the Masonic Fraternity, of Berkeley, Cal. He leaves a wife and four children, the oldest of whom, Augustus Victor Saph, Jun. Am. Soc. C. E., was one of his father's assistants and succeeds him in business.

Mr. Saph was elected an Associate of the American Society of Civil Engineers on October 1st, 1901; an Associate Member on June 4th, 1907; and a Member on October 29th, 1912.

ADOLPH EUGENE SCHNEEWEISS, M. Am. Soc. C. E.*

DIED DECEMBER 25TH, 1920.

Adolph Eugene Schneeweiss, the son of Christian and Marie Widman Schneeweiss, was born at Stuttgart, Germany, on November 18th, 1864. He received his technical education at the Royal School of Technology, Stuttgart, from which he was graduated in 1887, receiving the degree of Civil Engineer. During the summers, from 1879 to 1886, he was employed as Draftsman and Transitman on survey and construction work in the Department of Sewers and Canals at Stuttgart.

Shortly after his graduation, Mr. Schneeweiss came to the United States and entered the Engineering Department of the Erie Railroad in 1887. From this time until 1891, he was employed as Draftsman and Transitman on a wide range of work, including preliminary surveys for new lines, construction, maintenance work, and terminal and water-front improvements.

In 1891 he left the Erie Railroad and spent about eighteen months with F. L. Olmstead, as Draftsman and Topographer, on landscape engineering in connection with the development of a large estate at Biltmore, N. C. At the completion of this work in 1892, he joined John W. Ferguson, M. Am. Soc. C. E., Engineer and Building Contractor, of Paterson, N. J., as Engineer.

In 1913, Mr. Schneeweiss accepted the position of Vice-President of the Newark Cornice and Skylight Works, Newark, N. J., manufacturing fire-resisting doors and windows. He found this work uncongenial, and rejoined the John W. Ferguson Company in 1917, where he continued until his death. While with this Company he was engaged on the design and construction of industrial plants, many of which were of considerable magnitude.

With his training in design and construction, Mr. Schneeweiss combined a keen perception, at times almost intuitive, and great analytical power. These, with the judgment acquired from long experience, enabled him to solve satisfactorily the many varied problems constantly arising in connection with industrial design. His opinion on these matters was highly valued, and he was often called in consultation on valuation work and the remodeling of existing plants, as well as new design.

His fine personality, even more than his marked capabilities as an engineer, won for Mr. Schneeweiss the respect and affection of all with whom he was associated. He was modest, retiring, and generous to a fault, sacrificing his own personal interests whenever he might benefit others. His sudden death came as a great shock to all who knew him, and he will be greatly missed.

His first wife, Luise Motz, to whom he was married in 1893, died in 1910. In 1912, he was married to Emma Zerwick, who, with three daughters and a son, survives him.

Mr. Schneeweiss was elected a Member of the American Society of Civil Engineers on October 5th, 1909.

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* Memoir prepared by J. W. Ferguson, M. Am. Soc. C. E.

JAMES CRUICKSHANK SCORGIE, M. Am. Soc. C. E.*

DIED FEBRUARY 16TH, 1921.

James Cruickshank Scorgie, the son of George and Helen Scorgie, was born at Aberdeen, Scotland, on August 2d, 1850. After a common school education in which he showed promise in mathematics, he entered the employ of the Harbor Board at Aberdeen. He began in the Paymaster's Department and afterward was employed in the Engineering Department in the building of a breakwater.

He came to the United States in 1873 and began work in Cambridge, Mass., as Draftsman and Engineer for large granite builders. The erection of the State Insane Asylum at Worcester, Mass., and the Reformatory at Concord, N. H., were guided by detailed drawings made by Mr. Scorgie. In 1880 he accepted the position of Engineer at Mt. Auburn Cemetery, at Cambridge, later serving as Assistant Superintendent and, in 1896, he became Superintendent and held that position until the time of his death, making a continuous service of more than forty years.

His engineering duties in this connection involved the design and construction of ½ mile of 10-ft. concrete boundary wall; the construction and repair of miles of highway; the construction and repair of water supply and drainage systems; the design and construction of one of the first crematories built in this country; and the construction of the office and chapel building at the Cemetery.

At the time of his death, which followed a series of three operations that physicians expected would be of minor importance, Mr. Scorgie had received leave of absence from the Trustees of Mt. Auburn Cemetery and was entering on a period of well-earned rest. From his early youth in Scotland and all the time that he was employed as Draftsman and Engineer in the various undertakings, he improved himself by attending evening schools and by self-instruction during his spare time. He joined engineering societies, and also associations of men in charge of cemeteries. He became a close student of plant life, especially flowering shrubs and specimen trees, and was an active member of horticultural clubs in the vicinity of Boston. Besides the noteworthy qualities of close application and perseverance, Mr. Scorgie possessed true Scotch humor, and was always ready with joke and story. He was a member of Scottish societies, a Mason, and a Granger, and was an active participant in the Unitarian Laymen's League. He was very prominent in all town affairs at his summer home in Harvard, Mass.

He is survived by his widow, Mrs. Anna Monson Scorgie, and three daughters, Anna, Helen, and Elvira.

Mr. Scorgie was elected a Member of the American Society of Civil Engineers on November 4th, 1908.

* Memoir prepared by Frank B. Sanborn, M. Am. Soc. C. E.

DAVID CHAUNCEY SHEPARD, M. Am. Soc. C. E.*

DIED AUGUST 7TH, 1920.

David Chauncey Shepard was born on a farm a short distance from Geneseo, N. Y., on February 20th, 1828. He attended the district schools and Temple Hill Academy at Geneseo, and Brockport Collegiate Institute, at Brockport, N. Y. When nineteen years of age, he was appointed by Governor John Young as Assistant Engineer and Draftsman on the construction of the Genesee Valley Canal from Sonyea to Olean, N. Y. After four years of service, he resigned this position to engage in the surveys for the Dorchester and Genesee Valley Railroad—now a part of the Erie System.

After service on the Erie Canal and in the State Engineer's office at Rochester, N. Y., in the spring of 1852 Mr. Shepard was appointed Chief Engineer to take charge of the construction of the Canandaigua and Niagara Falls (now New York Central) Railroad, from Honeoye Falls, N. Y., to the crossing of the Genesee Valley Canal. This was the first experience in railroad construction of one whose achievements in that science are probably unequalled, either in variety or extent.

In 1853, Mr. Shepard began his career in the West, first as Chief Engineer of the Cleveland, Zanesville and Cincinnati Railroad, then as Division Engineer on the construction of the Atlantic and Great Western Railway (now a part of the Erie), and in 1854 he was appointed Chief Engineer of the road extending from Orangeville to Dayton, Ohio. In June, 1857, Mr. Shepard was appointed Chief Engineer of the Minnesota and Pacific Railway, now a part of the Great Northern System, and from that time until his decease he resided continuously in Minnesota. The first physical act in the construction of what is now the Great Northern Railway was accomplished by Mr. Shepard. In the chronicle of this event, Andrews' History of the United States contains the following statement:

"The pioneer railway in Minnesota was laid in 1862 from Saint Paul to Saint Anthony. The first shovelful of earth was lifted by a citizen of St. Paul, who, if he still lives, sees his State gridironed with 5 379 miles of track, his own firm constructing over 1 100 miles in the single year of 1887."

While Chief Engineer of the small railway in which the Great Northern System had its beginning, Mr. Shepard recommended an extension of the road a few miles farther north. The late James J. Hill, F. Am. Soc. C. E., a few years ago, published a report made by Mr. Shepard, replete with fact and argument, to demonstrate that, notwithstanding the severity of Minnesota winters, the fertility of the soil and the nature and variety of the products which would be offered for transportation were such that the extension would be justified. Over this portion of the Great Northern Railway, the commerce of an empire moves to-day.

In 1863, Mr. Shepard was appointed Chief Engineer of the Minnesota Central Railway Company, now a part of the Chicago, Milwaukee and St. Paul Railway, extending from Minneapolis to the Iowa State line. Afterward, he was made Chief Engineer during construction of all the lines in operation in Minnesota and Northern Iowa. In 1871, he resigned his position with the Chicago, Milwaukee

* Memoir prepared by M. V. Seymour, Esq. St. Paul, Minn.

and St. Paul Railway Company and its kindred enterprises to take charge of the affairs of the Northwestern Construction Company, then engaged in constructing the Northern Pacific Railroad from North Pacific Junction, Minn., to Fargo, N. Dak. With the completion of this contract in 1872, Mr. Shepard's active life as a Contractor commenced and was continued until 1894, when he retired from all business.

During Mr. Shepard's life he, and the firms of which he was a member, constructed more than 7 000 miles of railway in thirteen States of the United States and in the Canadian Northwest. In fifteen months, commencing May 1st, 1882, his firm built 675 miles of the Canadian Pacific Railway from Oak Lake to Calgary, in which there were moved 10 000 000 cu. yd. of earth, and all the timber and piling used in building bridges, culverts, and other structures in the roadbed were hauled ahead from the end of the track and placed in the work before the track was laid. While laying the track on this line, 6 miles and 600 ft. of track were laid in a single day. In 1887, his firm constructed 1 175 miles of railroad, of which the remarkably rapid construction of the St. Paul, Minneapolis and Manitoba Railway, from Minot, N. Dak., to Helena, Mont., formed a part. The track-laying on this great work was commenced five miles west of Minot, on April 1st, and was completed to Helena, on November 18th, 1887, a distance of 643 miles, and at an average rate for each working day of 3¼ miles. Of this distance, 500 miles remained, on April 1st, 1887, to be graded and bridged ahead of the track-layers, and this work was completed on September 1st, 1887, at the average rate of 100 miles per month. From the mouth of Milk River to Great Falls, Mont. (200 miles), the grading progressed at the rate of 7 miles per day.

In *Harper's Magazine* for March, 1888, there is an article entitled "Studies of the Great West", from the pen of Charles D. Warner, which describes at length this work. In speaking of the rapid progress of the grading, he says:

"Those who saw this army of men and teams stretching over the prairie and casting up this continental highway think they beheld one of the most striking achievements of civilization."

In commenting on the wonderful achievements of which Mr. Shepard was a witness, he made the following statement:

"My generation has been fruitful in all lines of invention and in their application to the comfort, well-being, and happiness of mankind. Steam, electricity, and the very air we breathe have been harnessed and made to do our bidding, so that we stand in awe and wonder when we view the results achieved. Space does not allow an enumeration, but I here express the doubt that any descendant of mine who reads these words can ever witness the application of so many and such wonderful inventions and discoveries for the quick transmission of matter, power, intelligence and sound to remote places, in his generation, as I have had the happiness to witness and enjoy in mine."

Mr. Shepard was married on December 24th, 1850, to Frances Aurelia Parsons, of Geneseo, N. Y. He survived his wife, son, and daughter.

He was a wonderful man. No one knew Minnesota history better than he, for he helped to make it. He was not only a great reader, but a great thinker. His mind, by reading and reflection, was kept receptive and keen to the last. His love

for his Profession—that of engineering—never faltered. During the last years of his life, he delighted in the modern methods of concrete construction, of which he made an earnest study. He was a man of culture. Although physically incapacitated, he enjoyed his books to the last.

Those whose privilege it was to know him best can never forget his high standards of honor and conduct, nor his kindness.

Mr. Shepard was elected a Member of the American Society of Civil Engineers on May 2d, 1883.

GEORGE STEELE SKILTON, M. Am. Soc. C. E.*

DIED JANUARY 9TH, 1921.

George Steele Skilton, the son of Dr. Avery Judd and Mary Augusta Candee Skilton, was born on November 25th, 1843, in Troy, N. Y. His preparatory school education was received in his home city, and in 1868 he was graduated from the Rensselaer Polytechnic Institute. During the Civil War he acted as aide to his brother, the late Dr. Julius A. Skilton, who served as a Surgeon with the Northern Army.

After graduation, Mr. Skilton went to the City of Mexico with his brother, who had been appointed Consul, and he himself later became Vice-Consul. From 1871 to 1886, he was engaged on railroad work, mainly in Mexico, becoming Chief Assistant Engineer in charge of construction of the Mexican Central Railroad in 1880, and in 1882 Chief Engineer of the first Southern Division of that railroad. He was General Superintendent and Chief Engineer of the Sinaloa and Durango Railroad in 1884.

In 1886, Mr. Skilton came to Brooklyn, N. Y., and was engaged in manufacturing as a partner in the Palmer Metal Manufacturing Company. In 1894 he was appointed by Mr. A. T. White, then Commissioner of City Works, Brooklyn, as Engineer in Charge of the Brooklyn water-shed system on the south side of Long Island. He remained connected with the water supply system of the city from that date until he retired in 1914.

He was married, in 1880, to Adah G. Sawyer, daughter of the late Jerome and Adeline Sawyer.

Mr. Skilton was always active in civic matters, and for more than thirteen years was a member of the Board of Education of Rockville Centre, Long Island. He was a member of the Volunteer Fire Department and for many years served as Treasurer of his company. At the time of his death he was President of the Library Board of Rockville Centre.

Mr. Skilton's long life leaves an enviable record of work accomplished both in his profession and as a citizen. The high place which he occupied in his community for clearness of vision and unselfish interest in the welfare of his fellow men, has been shown by the offices to which he was selected, and by the numerous public and private expressions of appreciation and esteem that were called forth by his death.

Mr. Skilton was elected Member of the American Society of Civil Engineers on September 7th, 1881.

* Memoir prepared by William W. Brush, M. Am. Soc. C. E.

EDMUND GYBBON SPILSBURY, M. Am. Soc. C. E.*

DIED MAY 28TH, 1920.

Edmund Gybbon Spilsbury was born in London, England, on December 7th, 1845. At an early age he was sent to Liége, Belgium, where he received his preparatory education. His technical education was gained at the University of Louvain, Belgium, from which he was graduated in 1862. After his graduation, Mr. Spilsbury went to Clausthal, Germany, for a practical course in his chosen profession, mining engineering and metallurgy.

In 1864, he entered the service of the Eschweiler Zinc Company, Stolberg, Germany, one of the largest mining and smelting companies of lead and zinc in the world, as Assistant Engineer. In 1865, he was sent by the company to the Island of Sardinia to take charge of its mines and works there, and, afterward, to the Atlas Mountains in Morocco. During his stay in Sardinia, Mr. Spilsbury entertained, for the company, the Duke of Brabant, afterward Leopold II of Belgium, who had come to the Island on a mountain sheep hunt, and formed a friendship that continued during the life of the King.

In 1867, Mr. Spilsbury entered the employ of McClean and Stilman, of London, England, and had charge of the construction of the iron gates for the Surrey Commercial Docks.

In 1868, he was engaged by J. Casper Harkort as Designing Engineer, and while in this position had charge of most of the detail work of the Keulenberg Bridge, in Holland, and of the Danube Bridge, in Vienna, Austria, as well as of the Rhine Bridge, at Düsseldorf, Germany.

In 1870, Mr. Spilsbury returned to the service of the Eschweiler and Corphalic Company as Chief Engineer, and while still in its employ came to the United States to investigate this country's resources in lead and zinc. He spent two years in this work, during which he explored the northern shore of Lake Superior and spent some time in exploration work in Colorado, Montana, Utah, and California.

In 1873, he resigned his position with the Eschweiler and Corphalic Company to engage in private practice in the United States as a Consulting Mining Engineer, and in the course of his practice, he introduced, for the first time in this country, the Harz system of ore-dressing for the zinc ores of Pennsylvania and New Jersey.

In 1875, Mr. Spilsbury was appointed General Manager of the Bamford, Pa., Smelting Works, which position he retained until 1879, when he resigned and built the Lynchburg, Va., Blast Furnace and Iron Works and also acted as Consulting Engineer for the Coleraine Coal and Iron Company, of Philadelphia, Pa.

In 1883, he was made General Manager of the Haile Gold Mine in South Carolina, and, in 1887, he entered the employ of Cooper, Hewitt and Company, of New York City, and was Managing Director of the Trenton Iron Company, Trenton, N. J., from 1888 to 1897. While in the latter position, he introduced as specialties of the company's business the Elliot locked wire rope and the Bleichert system of aerial tramways.

* Memoir compiled from information on file at the Headquarters of the Society.

From 1897 to the time of his death, Mr. Spilsbury was engaged in private practice as a Consulting Mining and Metallurgical Engineer. He had made for himself an international reputation and his practice took him into many parts of the United States, Europe, Africa, Mexico, and South America. Only a few weeks before his death, he had returned to the United States from work on a mining project in Brazil.

He died suddenly on May 28th, 1920, of heart failure, at the New York Eye and Ear Infirmary where an operation for cataract had been performed successfully on his eyes a few days before. He is survived by three sons and a daughter, and by a sister.

In 1893, Mr Spilsbury presided over the sessions of the Mining Division of the International Engineering Congress at the World's Fair, at Chicago, Ill.

He was a member of the American Institute of Mining and Metallurgical Engineers, having served as its President in 1896, the Mining and Metallurgical Society of America, the Institution of Mining and Metallurgy of Great Britain, the American Electrochemical Society, the American Society of Mechanical Engineers, and the Rocky Mountain Club and the Engineers' Club of New York City, of which latter club he was President in 1916 and 1917.

As representative of one or another of the National Engineering Societies, Mr. Spilsbury was a Trustee of United Engineering Society from 1916 until his death; a member of Engineering Societies Library Board from its organization in 1913 until 1920, having served as Chairman from 1918 until January, 1920; and a member of the Engineering Foundation Board from 1916 until his death. He was also a member of the Division of Engineering of the National Research Council of Washington and New York City.

Of all these societies and boards, Mr. Spilsbury was an active, useful member, serving on numerous committees and contributing freely of his time and ability.

He was the author of a number of technical papers and discussions, the most important of which are published in the *Transactions* of the American Institute of Mining and Metallurgical Engineers and the *Engineering and Mining Journal.* In 1915, he contributed an article on "Technical Reminiscences" which appeared in the *Mining and Scientific Press,* in which he recalled many of his early interesting experiences in Sardinia and elsewhere and the notable progress in mining engineering during his half century of active practice. Relative to the latter, the following extract may be of interest:

"I wonder how many mining engineers of the present day can call to mind changes and advances in the practice of mining which have taken place in the last half-century? Practically every appliance beyond the pick and shovel, hand-hammer and drill, has been introduced during that short period. When I first started my practical course in underground work, we knew none of the aids now considered requisite to work economically. We had no machine drills to save the physical labor of hand work. Dynamite or its precursor, nitro-glycerine, had not yet been invented; fuse was still unknown, even steel was more or less of a luxury, and our hand-drills were still made of iron with steel bits welded on to them. The mushrooming of the head of the drill under the hammer blows was a constant source of trouble and injury to the hands of the miners. The only explosive known was the large-grained black powder.

"It seems hardly credible to-day, and yet it is a fact, that the contract price for driving or stoping ground was influenced by the condition of the rye crop.

In seasons of drought, the height of the rye was much curtailed and the length of the straw between joints would be so short that the time occupied in jointing and filling the straws which were used as fuses, greatly lessened the driving capacity of the miners. The straw primers naturally limited the possible depth of the hole and 3 ft. was considered a good average. These straw fuses were very rapid and it was necessary to ignite them by a slow-burning sulphur wick.

"In the German mines the men all assembled in the foreman's office for prayers before beginning their shift. Except under special conditions no men were allowed to be raised or lowered by the hoisting engine, the ladders being used both for entry and exit. In some of the very deep mines, however, a special hand-engine was used, greatly lessening the time spent in climbing ladders."

Mr. Spilsbury was elected a Member of the American Society of Civil Engineers on December 7th, 1892.

ROBERT PARKER STAATS, M. Am. Soc. C. E.*

DIED AUGUST 18TH, 1920.

Robert Parker Staats was born in New York City, on August 23d, 1853. His ancestors, on his paternal side, came from Holland in 1629, and settled near Albany, N. Y. He was educated at Grammar School No. 35 and the College of the City of New York.

In 1871, Mr. Staats entered the office of Gen. Charles K. Graham, Civil Engineer, in New York City, to take up the study and practice of civil engineering. In 1874, Gen. Graham, who had succeeded Gen. George B. McClellan as Chief Engineer of the Department of Docks, of New York City, had Mr. Staats appointed as Surveyor for the Department, in which capacity he served until 1877.

In 1878, after a competitive examination, he was appointed Assistant Engineer on the Rochester Division of the Erie Canal. From 1880 to 1882, he was employed as Assistant Engineer for the Erie Railroad Company on bridges and structures, and from 1882 to 1884 he served as Engineer of the New York Division of the same road. In 1885, Mr. Staats was engaged as Engineer on Construction of the Norwood and Massena Springs Railway.

In 1886 he formed a partnership with Mr. Henry M. Doremus, for the purpose of engaging in private practice as a Constructing Engineer. Upon the retirement of Mr. Doremus, in 1888, Mr. Staats associated himself with his brother, John H. Staats, M. Am. Soc. C. E., and continued the business as R. P. and J. H. Staats. In 1892 the firm was changed to a corporation, Mr. Staats becoming President and occupying that position until his retirement and the dissolution of the company in 1915. The company specialized in harbor construction and built many of the large steamship and railroad terminals in and about New York City. Mr. Staats designed the steel work for the Tower Building, the first high office building in New York City.

He died at his summer home, Folly Farm, at Great Barrington, Mass., on August 18th, 1920, and is survived by his wife, a daughter, and a son.

He was a member of the Holland Society, the Union League Club, and the American Geographical Society, and a Veteran of the Seventh Regiment, N. G. N. Y.

Mr. Staats was elected a Junior of the American Society of Civil Engineers on November 3d, 1875, and a Member on July 1st, 1885.

* Memoir prepared by J. H. Staats, M. Am. Soc. C. E.

EMIL SWENSSON, M. Am. Soc. C. E.*

DIED MAY 13TH, 1919.

Emil Swensson, the son of Jean and Marie Katherine Swensson, was born in Aalborg, Denmark, on December 12th, 1858. He was one of a family of seventeen children.

He spent his early boyhood days in Halmsted, Sweden, and was graduated from Chalmer's Polytechnic Institute, Gothenburg, Sweden, in June, 1878, as a Civil and Mechanical Engineer. He then went to Germany to continue his studies and after spending nearly two years there, he was engaged by the Town of Warberg, Sweden, as Engineer in Charge of Harbor Improvements. Being intensely ambitious and desiring to follow his own course independently of his family, he came to America in August, 1880.

Mr. Swensson's early days in America were anything but cheerful ones. He found himself greatly handicapped by his lack of knowledge of the English language, and, for some time, he could secure no connection within his profession. Nothing daunted, he went to work at hard manual labor in the first projected tunnel under the Hudson River, applying himself meanwhile to the study of the language of his adopted land.

Mr. Swensson's first professional engagement was with Edward Raht, an Architect in New York City, as a Draftsman. On January 1st, 1882, he was engaged by the late O. W. Barnes, M. Am. Soc. C. E., Chief Engineer of the South Pennsylvania Railroad, as Assistant Engineer in charge of a locating party on that road. On October 31st, 1885, the construction of this railroad having been abandoned, he entered the employ of the Phœnix Bridge Company, under the late A. Bonzano, M. Am. Soc. C. E., Chief Engineer. In February, 1885, Mr. Swensson located in Pittsburgh, Pa., and made that city his home. His first engagement there was with the Keystone Bridge Company, C. L. Strobel, M. Am. Soc. C. E., Chief Engineer, which eventually became a part of the American Bridge Company. He remained with this company for sixteen years, rising from the position of Draftsman to that of Chief Engineer, then General Superintendent, and finally Manager of Works.

On January 1st, 1901, Mr. Swensson engaged in private practice, with offices in the Frick Building, Pittsburgh. On account of ill health, he retired from active work in 1917. During that period he was connected, as Consulting Engineer, with many important works of a wide range.

Within the Profession, Mr. Swensson was known as a close student, an advanced thinker, a strict disciplinarian, and a man of fine judgment. His bearing was dignified, but he was ever ready to join in the wholesome amusements of life. He was a man of engaging personality and obliging disposition who, starting his life-work alone and friendless, attached to himself a wide circle of friends. Measured by the results, his was a successful life.

He was married to Catherine Jordan, of Mt. Pleasant, Pa., on Christmas Day, 1883, and was the father of four children: Otto J., Christine, Stuart, and Henri.

Mr. Swensson was elected a Member of the American Society of Civil Engineers on June 7th, 1893. He served as a Director of the Society from 1906 to 1908, and held the office of Vice-President during 1909 and 1910.

* Memoir prepared by George S. Davison, M. Am. Soc. C. E.

EDWIN THACHER, M. Am. Soc. C. E.*

DIED SEPTEMBER 21ST, 1920.

Edwin Thacher was born in De Kalb, St. Lawrence County, N. Y., on October 12th, 1840, the youngest of four children and the only son of Dr. Seymour and Elizabeth Smith Thacher. Both his parents were of English stock through Colonial New England families, his father having been one of the leading physicians of St. Lawrence County for more than forty-five years. The family moved to Hermon, in the same County and State, about 1851 or 1852, when Edwin was quite young.

After taking a preparatory course of studies at the Wesleyan Seminary, Gouverneur, N. Y., he entered the Sophomore Class of the Rensselaer Polytechnic Institute at Troy, N. Y., in September, 1860, and was graduated as a Civil Engineer, with high honors, in the Class of 1863.

Soon after graduation, Mr. Thacher was employed as Assistant Engineer of the Cedar Rapids and Missouri River Railroad, in Iowa, of which the late William W. Walker, M. Am. Soc. C. E., of the Class of 1856, Rensselaer Polytechnic Institute, was Vice-President and Chief Engineer. He resigned from this position early in 1864 to become Assistant Engineer on the United States Military Railroads, Department of the Cumberland, with headquarters at Nashville, Tenn., and continued in this Government service until the close of the Civil War in 1865. Early in 1866 he accepted a position at Louisville, Ky., as Principal Assistant Engineer under the late Gen. I. M. St. John, M. Am. Soc. C. E., Chief Engineer, Louisville, Cincinnati and Lexington Railroad, in connection with the construction of the Cincinnati Branch, running from La Grange to Cincinnati, Ohio. When this work neared completion, he resigned to accept, in 1868, a position as Assistant Engineer of the Louisville Bridge Company, which was then constructing the 14th Street Bridge, one mile in length, over the Ohio River at Louisville, Ky. (now owned and operated by the Pennsylvania Railroad Company), remaining in this capacity until the bridge was completed and opened to traffic in 1870.

After severing his connection with the Louisville Bridge Company, Mr. Thacher entered the service of the Louisville Bridge and Iron Company, at Louisville, Ky., on August 1st, 1870, as Assistant and Computing Engineer, where he remained until June 11th, 1879, when he resigned to accept an appointment as Computing Engineer of the Keystone Bridge Company, Pittsburgh, Pa. He became Chief Engineer of that Company in 1883 or 1885, in which capacity he continued until October 13th, 1887, resigning to accept the position of Chief Engineer of the Decatur Bridge and Construction Company, Decatur, Ala., to which place he removed his family from Pittsburgh, on October 30th, 1887, and immediately assumed his duties as Chief Engineer. On May 2d, 1888, he was elected Vice-President and General Manager, and, on May 15th, 1888, was made Receiver of the Company when the shops were closed.

* Memoir prepared by A. W. Buel, M. Am. Soc. C. E., and F. H. Vaughan, President, Louisville Bridge and Iron Company, Louisville, Ky.

In the latter part of 1889, he moved from Decatur to Louisville, Ky., having closed his business there, and established a Consulting Engineer's office. This was continued until October 25th, 1894, when he formed a partnership with Messrs. Keepers and Wynkoop at Detroit, Mich. Mr. Wynkoop dropping out of the partnership on September 3d, 1895, the business of bridge designing and contracting was thereafter continued under the name of Keepers and Thacher until October 5th, 1899, when the partnership was dissolved.

Mr. Thacher then individually conducted a bridge designing and contracting business until May 1st, 1901, when he became associated with William Mueser, M. Am. Soc. C. E., under the firm name of the Concrete Steel Engineering Company, with headquarters in the Park Row Building, New York City. His connection with this Company continued until May 1st, 1912, when he retired from the firm and active business, after forty-nine years of continuous and successful practice of his profession, forty-four years of which had been devoted, mainly, to the design and construction of large bridge structures throughout the United States and, to some extent, in Porto Rico and Canada.

During the period from the latter part of 1889 to October 25th, 1894, when he was occupied as Consulting Engineer and Bridge Contractor at Louisville, Ky., Mr. Thacher designed and constructed bridges and viaducts for some of the leading Southern railroads to a value of about $2 000 000, and also many highway bridges, including that across the Willamette River, at Portland, Ore., the piers of which rest on piles driven and sawed off more than 100 ft. below the surface of the water.

From October 25th, 1894, to October 5th, 1899, while he was associated with Mr. W. H. Keepers at Detroit, Mich., under the firm name of Keepers and Thacher, this firm constructed the concrete steel arch bridge over the Kansas River at Topeka, Kans., at that time the largest bridge of its kind in the United States, as well as many other bridge structures of lesser note.

Mr. Thacher was a pioneer in the design and construction of concrete steel bridges in the United States, having made this type of structure a specialty since 1895. The greater number of the larger concrete steel arch bridges that have been built in the United States, up to the present time, have been designed by the concerns in which he was a prominent member.

In 1900-01 he contracted with the U. S. Government for the construction of two large concrete steel bridges in Porto Rico.

Mr. Thacher contributed many valuable papers to the Society, and participated, quite frequently, in the discussions at its meetings. He also occasionally delivered lectures at Cornell University and at other institutions in the United States. He was the inventor and patentee of the "Thacher Cylindrical Slide-Rule"; "Improved Duplex Slide-Rule"; "Thacher Steel Bridge Truss"; "System of Concrete Steel Arches"; "Steel Superstructure for Concrete Steel Bridges", and the author of works on the slide rule and concrete steel construction. He was also the inventor and patentee of the "Thacher Combination Bridge Truss" (practically free from thermal stresses), of the "Thacher Deformed Bar for Reinforcing Concrete", of "Concrete Steel Sheet Piling", of "Composite Piles of Wood and Concrete", and of a "Concrete Steel Floor System".

Among the foregoing specified patents, those for the "Combination Bridge Truss"; the "Cylindrical Slide-Rule"; the "System of Concrete Steel Arches", and the "Deformed Bar for Reinforcing Concrete" have been extensively used.

Mr. Thacher was keenly interested in all things tending to the world's progress, and was identified with many societies and organizations designed to promote the world's best development. Among these may be named the American Society for Testing Materials and the Rensselaer Polytechnic Institute Alumni Association of New York City. He entertained liberal religious views, but was never identified with any church organization. In politics he was always a steadfast Republican. He was a member of the Masonic Fraternity, affiliated with Hermon Lodge, No. 500, F. and A. M., of Hermon, N. Y., and of the St. Lawrence Chapter, No. 132, R. A. M., of Canton, N. Y. He was one of the most democratic of men in character and habit and possessed a personality that drew about him, and retained, friends. Loyalty to these friends was one of his prominent characteristics which endeared him to all his associates. He had an affectionate and appreciative nature and was endowed with a strong constitution, his prominent characteristics being an indomitable will and untiring energy of purpose, with an ambition to master any proposition or problem which presented itself.

During the last years of his life Mr. Thacher was afflicted with a cataract on each eye, both of which were operated on at different times, so that he never became entirely blind, although his sight was greatly impaired. The first operation was not entirely successful, but the second afforded him quite satisfactory sight, so that he could read and write and go about unattended. He was cheerful, good company, and enjoyed his friends, to whom he was strongly devoted. Had his eyesight not been impaired he probably would have continued in active practice to the end of an even longer life.

He was vigorous in mind and body for a man of eighty, and was still enjoying life when he died suddenly at his home in New York City, of angina pectoris, on September 21st, 1920, having been in his usual state of health up to this time. He was buried in the cemetery at Flushing, N. Y.

Mr. Thacher was married on April 22d, 1872, at Indianapolis, Ind., to Anna Elbertine Bartholomew, who died in New York City, on June 2d, 1905. He is survived by one child, Bessie, the wife of John C. Sample, Civil Engineer, of Flushing, N. Y.

The following extracts have been taken from a letter by B. H. Davis, M. Am. Soc. C. E., dated November 30th, 1920:

"Late in April, 1920, Mr. Thacher and the writer made an inspection of some of his work and during the course of our trip Mr. Thacher gave testimony as an expert witness in a case in which certain facts regarding his record as a Designing and Consulting Bridge Engineer were made by him under oath, some of which facts may be of interest.

"Mr. Thacher on the witness stand gave his age as 80, and stated his business or occupation to be that of Bridge Engineer, adding that from the year 1868 to 1912, covering a period of forty-four years, he had been engaged in active professional practice. During that time Mr. Thacher stated that he had designed at least 2 000 steel bridges and perhaps 200 concrete bridges. Of the 2 000 steel bridges he had designed, he stated that at least 1 000 were built. While making it plain that he had not superintended the construction of all the structures, he gave

specific evidence in a very large number of cases to show his intimate personal relationship to them and their construction problems.

"In answer to the question: 'Who designed the first large reinforced concrete bridge in the United States?' Mr. Thacher replied that he did, and named the bridge over the Kansas River at Topeka, Kans., known as 'The Topeka-Kansas Bridge', as the largest bridge of reinforced concrete that had ever been constructed up to that time.

"In answer to the question: 'Who made the first elastic theory design of a reinforced concrete bridge actually built in the United States?', Mr. Thacher replied that he believed he did, and again named the Topeka, Kans., structure, qualifying his remark by saying that some professors had gotten out some theories that could not be followed, and stating that he developed a really practical theory and made the first practical application of the elastic theory to reinforced concrete bridge design in the United States.

"As an active, busy, practical Bridge Engineer and Constructor of wide experience and great capacity for concentration and hard work, Mr. Thacher, in my opinion, stands quite alone in a class by himself as an instructor and developer of practical and efficient bridge engineers and designers. All engineers, young and old, who were ever associated with Mr. Thacher or sufficiently acquainted with him to consult and counsel with him, will unquestionably acknowledge a deep and lasting debt of gratitude. No problem or perplexity, no matter how trivial or how serious, was ever brought to his attention without receiving his very courteous, honest and careful consideration. His sympathetic appreciation of the cares and worries of other engineers, who without hesitation brought their troubles to him, will long be remembered by the host of friends who have received comfort and advice freely and without stint from his wide and rich experience covering practically every conceivable problem encountered in the practice of bridge engineering.

"Mr. Thacher's appreciation of materials of construction, their strength and durability might well be emulated by every engineer and designer. It grieved him sorely to see a splendid structure neglected and deteriorating through lack of ordinary care in keeping the structure clean or painted, and his satisfaction and delight in finding structures that he had built 30 to 40 years ago still in good condition and showing every indication of capacity and excellence of construction was beautiful to behold, and an inspiration. Neglect and misuse of materials of construction were, in his opinion, equivalent to abuse and cruelty to the same extent that such terms are applied to flesh and blood.

"The frankness, candor and point blank honesty of the great engineer and gentleman that he was, under any and all conditions and circumstances, always contending for the absolute truth with all the enthusiasm of the scientist and scholar, marked Edwin Thacher, at least in the hearts of all who had the pleasure and rare opportunity of knowing him, as one of the truly great and accomplished members of the American Society of Civil Engineers, and a very great credit and honor to the Society and to the Profession of Engineering."

The following tribute was contained in a letter from C. L. Strobel, M. Am. Soc. C. E., under date of October 9th, 1920:

"Mr. Thacher was well liked by his associates for his genial and kindly qualities, and he was very competent in his work. He had a special fondness and aptitude for calculations and tabular work, and he was very quick and reliable."

Frank C. Osborn, M. Am. Soc. C. E., in a letter dated October 19th, 1920, writes as follows:

"While I had corresponded with Mr. Thacher for several years, my first meeting with him was in March, 1885, when I assumed the position of Assistant Engineer under him as Chief of the old Keystone Bridge Company. I remained with him two years, learning more and more to admire him as an engineer, and to love him as a

man. He was kindness itself, always considerate, and enjoyed assisting deserving young engineers. During my many years acquaintance and association with him, there was never an unpleasant word between us."

Mr. Mueser, who was associated with Mr. Thacher in the Concrete Steel Engineering Company, contributes the following:

"Edwin Thacher was a fine mathematician; extremely thorough and careful in his calculations and conclusions. He loved to solve practical mathematical problems related to engineering, and would often sit nights until the early hours of the morning to find the solution of a problem, rather than lay the matter over for another day. It is not surprising, therefore, that early fame came to him through the invention and perfection of the Thacher Cylindrical Slide Rule, which he patented under date of November 1st, 1881, and which, to the present day, is extensively used in many engineering offices.

"He originated the Thacher Reinforcing Bar—the first mechanical bond bar proposed, made into final shape by direct rolling process.

"While a man of scientific thoroughness in engineering matters, Mr. Thacher was unpretentious, modest and almost shy in disposition. He was a generous friend at all times and ever willing to be helpful to those who came to him for assistance."

Mr. Thacher had the devoted attachment of those of his friends and associates who knew him well, on account of his sterling character, and received the admiration of those who were acquainted with his professional attainments. No one could see much of him without noting the fine characteristics of the man and his talents, as he was absolutely frank.

With a high sense of honor and straightforwardness, he seemed to be naturally endowed with a most charitable disposition, in the broadest sense of the term. He was generous to a fault, and never even complained of or resented the occasional impositions of acquaintances of weaker mould. He was very democratic and liberal in his opinions. Although an indefatigable worker, he enjoyed recreation and the society of his friends. He seemed to prefer to be the friend, companion, and co-worker of his subordinates and engineering employees, as far as practicable, rather than the "boss" or "chief."

His life and the vigor he still possessed in his eighty-first year is a good demonstration that hard work does not injure a man, since Edwin Thacher did not seem to think he was working unless he was strenuously occupied for more than 72 hours a week—week in and week out. One thing, however, he never did, and that was to worry about his work or business; for one reason, no doubt, because he never let it get behind.

As a Civil Engineer, Mr. Thacher was very talented and a clever designer. He was often quite ingenious, as is attested by his numerous patents, a number of which came into general use. Probably he will be longer remembered in connection with "Thacher's Calculating Instrument or Cylindrical Slide Rule" than by any of his other works.

He expended great amounts of labor and money in the preparation of tables for estimating standard structures during a period of over twenty-five years. Most of these became obsolete, from one cause or another, others he revised at later periods, only to see them again go out of date. One set of tables, however, entitled "Panel Weight Multiples or Stress Coefficients" consists of nineteen large sheets and is a set of influence tables for some thirty-four forms of fixed trusses (with the

shear multiples given for trusses with parallel chords, permitting the use of the tables for varying proportionate depths), plate girder swing spans without floor-beams and with two to seven panels each side of the center, nine forms of trusses, for swing spans with three to ten panels each side of the center, and for both center and rim-bearing spans, and two forms of "bob-tail" swing spans. This set of tables still retains its value.

As has been the case so often, it is quite probable that the most important and far-reaching work of Mr. Thacher has been that almost intangible but very real and powerful personal influence on younger engineers with whom he came in contact, many of whom have had a keen appreciation of an inspiration of high standards and have passed on to others, often anonymously, some of that received from him. Examples of such influence are difficult to segregate and defy evaluation, but many engineers of distinction would testify to its existence as a live force.

Specifically, this influence made, and still makes, for systematic methods and forms for calculations and estimates, every item in order and intelligibly recorded, with computations reduced to progressive tabular forms wherever possible, and for rational design in which each detail, however small, is subjected to the same rigid analysis and careful consideration as the larger problems of the structure or truss as a whole.

During all the successive periods when the timber Howe truss, the combination truss (wood in compression and wrought iron in tension), cast and wrought iron in combination, all wrought iron, structural steel and reinforced concrete were the available materials for the bridge builder, Edwin Thacher was actively engaged in designing and constructing bridges and exerted a much greater influence on the development of American bridge building practice than is generally appreciated. His influence has always been for rational and scientific design, combined with practical methods and details. No complete history of bridge building in America can fail to give him a large and important place.

Mr. Thacher was elected a Member of the American Society of Civil Engineers on February 17th, 1869.

ELLIS DUNN THOMPSON, M. Am. Soc. C. E.*

DIED FEBRUARY 9TH, 1920.

Ellis Dunn Thompson was born at Newark, N. J., on October 6th, 1854, and after receiving his preliminary education, was graduated from the School of Engineering, Cornell University, in 1876, and read law from then until 1879. He was connected with the Engineering Department, United States Army, from 1879 to 1904, most of this time as Assistant Engineer. During this period he was with the Mississippi River Commission on construction of river improvements for three years; with the Engineer Office at Beaufort, N. C., for three years, in charge of dredging and jetty construction, and acted as Principal Assistant Engineer at Wilmington, N. C., with supervision of all works of improvement in North Carolina.

From 1894 to 1904 he served as Principal Assistant Engineer in the Philadelphia District, where he had charge of the reconstruction of Philadelphia Harbor, which involved dredging of 43 000 000 cu. yd. of material, filling League Island, Petty Island, etc., costing approximately $15 000 000, the construction of the breakwater at the entrance of Delaware Bay involving 1 500 000 tons of stone at a cost of $2 350 000, and the improvement of the Delaware River involving a great amount of dredging, jetty construction, bulkheads, etc.

From 1904 Mr. Thompson was engaged in private practice as a Consulting Engineer on river and harbor work, having been connected with many large projects, until the United States entered the World War, when he became Engineer in Charge of Dredging for the Emergency Fleet Corporation, on which work he was engaged at the time of his death.

In this capacity he covered all the shipyards under the Emergency Fleet Corporation and had 140 dredging plants operating under his direction. It was one of the biggest pieces of work of this kind ever handled by one man in this country and involved unremitting labor on his part. Mr. Thompson probably had direct charge of more dredging than any man in the United States and his name was connected with many of the larger waterway improvements.

He was a man of great mental capacity and large attainments. He was a member of the Engineers Club of New York, Cornell Society of Civil Engineers, and the University Club of Philadelphia.

Mr. Thompson was elected a Member of the American Society of Civil Engineers on February 1st, 1899.

* Memoir prepared by John W. Thompson, Esq., Trenton, N. J.

ALFRED THOMAS TOMLINSON, M. Am. Soc. C. E.*

DIED JANUARY 20TH, 1921.

Alfred Thomas Tomlinson was born on June 22d, 1859, at Grand Falls, New Brunswick, Canada, where his father, Joseph Tomlinson, one of Canada's most eminent engineers, was engaged on some bridge construction. His parents were of English birth, but they spent over half a century in the United States and Canada, living where Mr. Tomlinson's professional engagements called him.

Mr. Tomlinson received his technical education at the Royal Military College, Kingston, Ont., Canada; and he twice served his country in a military capacity, first in the suppression of the Northwest Rebellion in 1885, and in the World War. Immediately on the declaration of the latter, he tendered his services with a request that he be sent to France, but the authorities decided that his age (55) was too great for the strenuous life of the battlefield, consequently, much to his disappointment, he was detailed, as Major of Canadian Engineers, to serve in Canada in charge of inspection of small-arms ammunition in Dominion arsenals. In this capacity he rendered valuable service to the Allied Cause.

Major Tomlinson's professional record is as follows: From 1885 to 1890 he was employed on railway construction in the Western States, including the position of Resident Engineer in charge of the construction of the Pike's Peak Cog Railway, also that of Resident Engineer on parts of the construction of the Denver and Rio Grande Railway.

From 1890 to 1893, he served as Assistant Engineer on the Brooklyn Elevated Railway, in charge of litigation surveys and of the construction of 1½ miles of elevated railway structure (track, stations, etc.) for steam operation; and from 1893 to 1894, as Engineer in charge of a section of underground electric railway for the Metropolitan Railway, New York City.

From 1894 to 1897, he was engaged as Assistant Chief Engineer, in charge of construction, on the Lake Street Elevated, (repairing and electrifying), the Northwestern Elevated, and the Union Loop Elevated Railways, in Chicago, Ill., on construction and electrical equipment; and from 1898 to 1901, he was Assistant Chief Engineer on the Boston Elevated Railway, Boston, Mass., in charge of construction including subway entrance, third-rail system, multiple-unit equipment on cars, etc.

From 1902 to 1905, as Civil Engineer for J. G. White and Company of New York City, he located and partly built the high grade interurban line from Rochester to Lockport, N. Y., the Marion and Wabash Railway in Indiana, and the Alton and East St. Louis Railway in Illinois.

From 1905 to 1908, Major Tomlinson was employed as Engineer for the Canadian White Company, at Montreal, Que., Canada, the Canadian Branch of J. G. White and Company, of New York City. The chief electrical work done by him while with this company was the construction of the hydro-electrical development of the surplus water of the Soulange Canal, at Cedars, Que. Among other work was 140 miles of construction for the Grand Trunk Pacific Railway, from Torchwood Hills to Saskatoon, and the trunk sewer for the City of Edmonton, Alberta.

* Memoir prepared by J. A. L. Waddell, M. Am. Soc. C. E.

From 1908 to 1912, he served as District Engineer, on the Grand Trunk Pacific Railway Company, representing that company on 400 miles of the Trans-Continental Railway, Sections C and D, of which the Grand Trunk Pacific Railway Company had the contract for 150 miles. This work was carried out by Foley, Welsh, and Stewart, Contractors, under Major Tomlinson's supervision, and during one year he had entire charge for the Contractor.

From 1912 to 1914, as Chief Engineer, he was in charge of the surveys and location of the North Railway, a proposed road from Montreal to James Bay; and from 1915 to 1919 he served as Major of Canadian Engineers, in charge of the inspection of small-arms ammunition in Dominion arsenals.

During some five or six years of his professional career, Major Tomlinson was connected with large constructions of which the writer was either in charge or Advisory Engineer, namely, the elevated railroads of Chicago, Ill., and Boston, Mass., consequently, he can vouch for the unusual thoroughness of Major Tomlinson's work, and for the characteristics of thoroughness, faithfulness, loyalty, and executive ability with which nature had endowed him to an eminent degree; and as the writer had known him intimately during a period of forty-five years, he takes deep satisfaction in bearing witness concerning his high character as a man, a gentleman, and an engineer.

Major Tomlinson was a Thirty-second Degree Mason, a member of the Iowa Consistory, No. 2, at Cedar Rapids, Iowa, and a member of the Rameses Temple, Mystic Shrine, Toronto, Ont., Canada. He was also a member of the Engineering Institute of Canada and the American Railway Engineering Association.

He was married to Miss Fannie Smith, of New York City, on May 10th, 1892, and Mrs. Tomlinson and their three children, Helen, Alfred, who served with the Canadian troops throughout the World War, and Myra, survive him.

Major Tomlinson was elected a Member of the American Society of Civil Engineers on September 7th, 1887.

WILLIAM MARTIN TORRANCE, M. Am. Soc. C. E.*

DIED MAY 18TH, 1920.

William Martin Torrance, the second son of S. Clay Torrance, was born in Persia Village, Gowanda, N. Y., on April 4th, 1874. His early education was obtained in the Gowanda District School, after which he prepared for college at the Gowanda Union School and Academy, from which he was graduated in 1890. Entering Cornell University in the fall of 1890, he took up the course of Civil Engineering until the summer of 1892. Remaining out of college for a year, he returned in September, 1893, and was graduated from Cornell University on June 20th, 1895, with the degree of Civil Engineer.

His first experience after leaving the University was with F. W. Wing, Consulting Engineer, of Buffalo, N. Y., and then as Engineer for the Gowanda State Homœpathic Hospital. In July, 1895, he joined the Engineering Staff of the Cleveland, Cincinnati, and St. Louis Railroad, advancing by promotion to the grade of Assistant Division Engineer with headquarters at Cleveland, Ohio. While officiating in this capacity he supervised the designing and laying out of all betterments, new yards, industrial tracks, re-alignments, and masonry structures, as well as the theoretical and field investigations of all bridges of the railroad for renewals, repairs, and replacements.

In July, 1898, Mr. Torrance was appointed by the Osborn Engineering Company of Cleveland, Ohio, as Designing Engineer of all steel bridges and viaducts fabricated by the Company. In June, 1899, he accepted an appointment with the Lehigh Valley Railroad as Assistant Division Engineer, and was assigned in charge of track layout and the concrete lining of the Musconetcong Tunnel of the New Jersey Division. During this engagement he devised a method of gauntleting tracks so as to permit the interlining construction and traffic to continue simultaneously.

In July, 1900, becoming associated with the bridge forces of the Chicago and Northwestern Railroad as Designing Bridge Engineer and then as Constructing Engineer, he had charge of the 1 500-ft. Wisconsin Railroad Bridge, at Grand Rapids, Mich., and the Fox River Bridge, at Princeton, Wis., and on their completion was put in charge of all classes of structures of that railroad.

In June, 1902, Mr. Torrance came East and accepted an assignment with the C. W. Hunt Company, of New Brighton, N. Y., as Designing and Estimating Engineer on coal-handling outfits, from which Corporation he severed his connection in February, 1903, and joined the engineering forces of the New York Central and Hudson River Railroad Company as Designing Engineer of the large masonry structures necessary in the improvement of the line along the east bank of the Hudson River in the suburbs of New York City. In May, 1903, he was appointed Resident Bridge Engineer of the Illinois Central Railroad in charge of the design and construction of the 1 500-ft. steel and reinforced concrete bridge over the Tennessee River at Gilbertsville, Ky., after the completion of which he returned to the New York Central Railroad and remained until July, 1905, at which time he accepted appointment with the Hudson River Tunnels Company, first as Assistant

* Memoir prepared by J. Springer Swindells, M. Am. Soc. C. E.

Engineer in charge of reinforced concrete design, and then as Superintendent of concrete construction of the Hoboken Terminal and Hoboken wye intersection.

At the New Jersey shore of the Hudson River where the tubes branch up and down the river, it became necessary to build short sections of tunnel as pneumatic caissons. As conditions were such that steel could not be secured within a year after the designs were finished, Mr. Torrance requested permission to design the caissons of reinforced concrete, which permission was granted. The completed plans were submitted to the Consulting Engineers for examination, and after approval he was appointed Construction Engineer to carry out his own designs. In all, five caissons were built above ground and sunk through the ground about 80 ft. to their proper level, about 70 ft. below tide level. The use of this idea saved the Hudson Company about $500 000 and enabled the tunnels to be opened six to eight months sooner than would otherwise have been the case.

After fourteen years of practical and theoretical experience, covering as it did achievements in the lines of the statics of the higher structures of engineering, Mr. Torrance in 1908 opened offices in New York City as a Consulting Engineer, under the firm name of Torrance and Taylor. This was the personnel of the firm until 1912, when the name was changed to Torrance and Swindells, and as such continued until the beginning of the World War.

During the period of his consulting practice many notable reports, investigations, designs, and constructions were undertaken for corporations, contractors, municipalities, States, and Governments, and included difficult foundations, reinforced concrete viaducts, bridges, warehouses, mechanically equipped manufacturing plants, wharves, power plants, and very large marine derricks, notable among which may be cited his connection with the following structures: Woolworth and Hallenback Buildings of New York City; Dry Dock No. 4, at the Brooklyn Navy Yard; Midland Linseed Products Buildings at Edgewater, N. J.; viaducts at Westport, Conn., Providence, R. I., Wilkes-Barre, Pa., Beaufort, S. C., Woodbine, Ga.; and reinforced concrete gun-cotton tanks and factories for the Russian Government. In 1915, Mr. Torrance opened additional offices in Savannah, Ga., and also acted as Chief Engineer of the Savannah Engineering and Construction Company. During this period many reinforced concrete structures were designed and built throughout the South, including wharves at Savannah, elevated reinforced concrete ore bins throughout Florida, and structural steel and concrete buildings throughout Georgia. While here, he secured patents on reinforced concrete bulkheads and elevated water towers.

In 1916, Mr. Torrance was instrumental in the organization of the Carson Construction Company of Savannah, Ga., of which Company he became General Manager, Vice-President, and Chief Engineer.

With the entry of the United States into the World War, his patriotism brought forth a ready response to his country's need and he presented himself for examination, was commissioned a Captain of Engineers, and was assigned to the Bridge Battalion of the 23d Engineers, located at Camp Meade, fully expecting to entrain with his regiment as part of a combat division. In November, 1917, he was assigned as Assistant to the Constructing Quartermaster in the design, development, and construction of the Raritan River Ordnance Depot at Metuchen, N. J., where he remained until July, 1918, when he resigned from the Service.

Becoming Assistant to the Chief Engineer of the Alphons Custodis Chimney Company, of New York City, he was assigned to the design and development of brick aerial towers of heights up to 2 000 ft. The result of his work in this capacity may best be described in the words of the Chief Engineer: "His knowledge and familiarity with the mathematics and governing factors necessary in the development of the statics of structures of these magnitudes was profound. He developed the most rational theory and the most economical commercial design of towers from 1 000 to 2 000 ft. high, that I know of; in fact, it was his design that made us low bidder on the U. S. Navy aerial towers that were being considered at the close of the war."

Returning to Savannah in 1919, Mr. Torrance continued as Chief Engineer of the Carson Contracting Company up to the time of his death on May 18th, 1920.

In personal characteristics, Mr. Torrance was a man of character, culture, refinement, and ability, genial, extremely generous, conscientious, and sincere. His calm and unruffled personality were important characteristics; he was always under firm self-control, even under the most exasperating circumstances; the most annoying and troublesome matters were settled fairly, justly, and without malice and he had a winning smile and a kind word for every one.

He was an entertaining companion in society, was well versed in literature and current topics, and generally entered into repartee. By his gentle persistence in avoiding needless controversy he was enabled, by personal effort, to put through many notable structures where an openly aggressive personality might have failed. This was especially noticeable in connection with the McAdoo Tunnels, the reinforced concrete foundations for the New York Navy Yard Dry Dock No. 4, the Hallenback Building on Lafayette Street, New York City, and the Wilkes-Barre Bridge.

He was extremely modest in personal demeanor, yet courageous in stating his position on any controversial subject, and he persistently followed fundamental truths, facts, and law in all his activities. As an engineer he was progressive, original, logical, and accurate in all deductions, indeed it might be said that in the line of statics he preceded current engineering practice as regards adaptation of engineering fundamentals and principles and, in addition, he was absolutely free from bondage to precedent.

Mr. Torrance was one of the earliest engineers to design reinforced concrete arch bridges as cantilevers with suspended spans, but the conservatism of municipal authorities postponed their introduction for years, owing to the design being considered too radical a departure from precedent. This method of design was evolved to compete with the Melan or Thacher type of arch.

As a classmate and associate, he was held in the highest esteem by all, from the Freshman entering college to the highest member of the Faculty, and not only was this fraternal feeling felt by fellow students and business associates, but it was also very pronounced in his brothers and sisters who passed through the portal of Cornell and who owed to him that conscientious support so necessary in securing the fundamentals on which to rear their future noble structure of accomplishment.

As a writer, Mr. Torrance was a frequent contributor to the engineering and scientific periodicals of this country, and as a material reward for his ability as an engineer and his contributions to the professional world for achievements of

notable exception, in 1909 he was awarded the Fuertes Graduate Gold Medal by Cornell University for his article on "Concrete Structures in the Hudson River Tunnels".

In 1907, he was elected an alumnus member of Sigma Xi and Tau Beta Pi. He was a member of the Western Society of Engineers and the Rotary Club of Savannah, Ga.

He was married in April, 1905, to Louisa Mitchell, of Chicago, Ill., and she and their three children, Philip M., Myra H., and Charles M., together with his father, S. Clay, brothers, Stiles A., Chester C., Robert S., and a sister, Rachael D., survive him.

Mr. Torrance was elected a Member of the American Society of Civil Engineers on July 1st, 1909.

BEVERLY REID VALUE, M. Am. Soc. C. E.*

DIED JUNE 10TH, 1920.

Beverly Reid Value was born at Montgomery, Ala., on April 7th, 1863. He was the son of Jesse R. and Edyth (Bailey) Value. He was graduated from the School of Mines, Columbia College, in 1884.

From 1885 to 1900, with the exception of one year, 1891-92, when he was engaged as Contractor on the removal of rock from the Passaic River at Little Falls, N. J., Mr. Value worked with the Aqueduct Commissioners, New York City, on the Engineering Corps for the first year or so. From 1886 to 1890 he was Assistant Engineer in Charge of Section 6 (until completion) of the New Croton Aqueduct, and from 1890 to June, 1891, he was engaged on final estimate work and the preparation of records for Court action. From March, 1893, to June, 1900, he was in charge of the construction of New Croton Dam, under the late Charles S. Gowen, M. Am. Soc. C. E., Division Engineer.

In June, 1900, Mr. Value joined the Rapid Transit Commission as Engineer, under William Barclay Parsons, M. Am. Soc. C. E., Chief Engineer, of the Third Division, extending from 104th Street and Broadway on the West Side, and the Lenox Avenue Extension on the East Side, across the Harlem River through 149th Street to the beginning of the elevated extension at Brook Avenue.

In 1903, Mr. Value resigned from the Rapid Transit Commission to become Chief Engineer of the Electric Development Company, and had charge of the construction of a large hydro-electric plant at Niagara Falls, N. Y. On the completion of this work in 1906 he became Executive Engineer of the Empire Engineering Corporation, of which his former chief, William Barclay Parsons, was President. This company, under the direction of Mr. Value, built several sections of the New York State Barge Canal, the plant of the McCall's Ferry Power Company, on the Susquehanna River, and other structures.

When the Empire Company and the H. S. Kerbaugh Company were consolidated as the Kerbaugh-Empire Company, Mr. Value became Comptroller and Director, and was in charge of the construction of the Kensico Dam of the Catskill Water Supply. Owing to the remarkable organization on the work, the Kensico Dam was practically completed three years ahead of the date required by the contract. Records were established for placing cyclopean masonry in dams of this height, 489 750 cu. yd. being placed in one season and 84 450 cu. yd. in one month.

Mr. Value subsequently became Vice-President of the Empire Engineering Company, Incorporated, which succeeded the Kerbaugh-Empire Company, and this position he held up to the time of his death. He was also Secretary, Treasurer, and a Director of George W. Rogers and Company. In the last few years his companies were engaged in barge canal and canal terminal contracting, and a large amount of railroad construction in Baltimore, Md.

Mr. Value was elected Treasurer of the General Contractors' Association in 1918, First Vice-President in 1919, and President in January, 1920. He was also a member of the American Institute of Mining and Metallurgical Engineers,

* Memoir prepared by J. Waldo Smith, M. Am. Soc. C. E., and F. B. Rogers, Esq.

National Geographic Society, Psi Upsilon Fraternity, the Engineers Club, of New York, the Columbia University Club, the Hartwood, N. Y., Club, and the Elizabeth, N. J., Town and Country Club.

His sudden death, after an operation which was not considered serious and which was deemed entirely successful, was a severe shock to the multitude of his friends and acquaintances. A polished gentleman, considerate and courteous almost to a fault, upright in all his dealings, and of extraordinary ability, he had a special faculty of making and keeping friends.

He is survived by a widow, two daughters, and two sons, one of whom, Beverly Mason Value, was Captain in the 21st Engineers, A. E. F., was awarded the Croix de Guerre by the French Government, and received two citations for conspicuous bravery and daring.

Mr. Value was elected a Member of the American Society of Civil Engineers on January 2d, 1901.

EDGAR BEACH VAN WINKLE, M. Am. Soc. C. E.*

DIED APRIL 27TH, 1920.

Edgar Beach Van Winkle was born in New York City on March 4th, 1842, son of Edgar Simeon and Hannah (Beach) Van Winkle. His ancestors on his father's side came from Holland prior to 1636 and settled in New York City, and on his mother's side from pure English stock. He was graduated from Union College with the degrees of A. B. and C. E. in 1861, and was immediately employed by the Croton Aqueduct Department on the New Grand Reservoir in Central Park, New York City.

At the outbreak of the Civil War, Mr. Van Winkle entered the Army of the United States as a Private, and was soon promoted to Lieutenant, and to Captain in 1868. He served on the staffs of Gen. Viele and of Gen. Q. A. Gillmore, M. Am. Soc. C. E., and was Chief of Staff of Gen. Hatch. During his Army service he was located chiefly in the eastern part of the Southern States.

At the end of the war, Colonel Van Winkle resumed the practice of his profession, and was employed in the Bureau of Sewers of the Croton Aqueduct Department, on the Erie Railroad, and on the Shepaug Valley Railroad. He was a member of the firm of Croes, Church and Van Winkle and was Chief Engineer of the Department of Public Parks, New York City, from 1878 to 1884.

He was elected a Companion of the Military Order of the Loyal Legion in 1866, and was Recorder of the New York Commandery of that Order and of the Commandery in Chief.

Colonel Van Winkle served in the National Guard of the State of New York as Colonel and Division Engineer of the First Division from 1876 to 1883. After 1885, his time and attention were devoted to affairs and business other than engineering.

He was married to Elizabeth, daughter of Judge William Mitchell, on June 7th, 1876, who died in 1894, and to Mary Flower, daughter of William Speiden, Esq., of New York City, on June 3d, 1899, who, with the children of his first marriage, three daughters and two sons, Edgar Beach Van Winkle, Jr., an architect and William Mitchell Van Winkle, a lawyer, survives him.

Colonel Van Winkle was always a courteous gentleman and a kind and generous friend.

Colonel Van Winkle was elected a Member of the American Society of Civil Engineers on December 2d, 1868, and served as a Director in 1880.

* Memoir prepared by George S. Greene, Jr., M. Am. Soc. C. E.

GEORGE WASHINGTON VAUGHAN, M. Am. Soc. C. E.*

DIED JUNE 5TH, 1920.

George Washington Vaughan was born on November 11th, 1859, at Paucatuck, Conn. He was educated at the public schools and at Warren Polytechnic College, at Providence, R. I.

In 1881 Mr. Vaughan entered railroad service as a Rodman on the Pittsburg, Cincinnati and St. Louis Railway, and was, successively, Assistant to Engineer, Maintenance of Way, and Supervisor of Track, until 1887. He then joined the New York, Chicago and St. Louis Railroad, and worked through the positions of Assistant to Division Engineer, Division Engineer, and Engineer in Charge of Maintenance of Way up to 1898. From 1899 until the time of his death, he was connected with the New York Central Railroad Lines, as Supervisor of Bridges and Buildings, Division Engineer, and Engineer of Maintenance of Way; in this last position he had charge of the upkeep, renewals, and maintenance of many miles of track, including very high-grade track over which daily limited express trains make maximum speeds.

Mr. Vaughan was married on February 18th, 1891, to Miss Fanny S. Minton, at Westfield, N. Y.

He died at Mount Vernon, N. Y., on June 5th, 1920, and is survived by a widow, one daughter, and three sons. Two of his sons were engaged in the activities of the World War.

Mr. Vaughan was a man of sterling integrity, of very pronounced personality, and was ever attentive to the interests of the railroad as they appealed to him.

Mr. Vaughan was elected a Junior of the American Society of Civil Engineers on March 3d, 1886, and a Member on November 4th, 1896.

* Memoir prepared by George W. Kittredge, M. Am. Soc. C. E.

LOUIS BERTRAND VAUGHAN, M. Am. Soc. C. E.*

DIED JULY 22D, 1920.

Louis Bertrand Vaughan, the son of Andrew Jackson Vaughan and Esther Norris Vaughan, was born in Boston, Mass., on August 22d, 1858. He came of New England ancestry and was a lineal descendant of George Vaughan who held the commission of Colonel in the Provincial Forces during Queen Anne's War, and in 1715 was appointed Lieutenant-Governor of the Province of New Hampshire. He received his early education in a private school in Providence, R. I., and in 1878 was graduated from the English High School.

Mr. Vaughan began his engineering work in the City Engineer's Office in Providence, and from 1878 to May, 1886, served as Rodman, Instrumentman, and Assistant Engineer in the City Property, Sewer, and Highway Departments, on surveys, and on the construction of sewers and roads. For a short time after this, he was engaged in private practice as a Civil Engineer in Providence, but gave up his office in May, 1887, to take up cable railroad work. In this special field he acted as Assistant Engineer for the Cable Tramway Company, of Omaha, Nebr., first in immediate charge of the construction of a power-house and the installation of the driving plant, and during the second season, in charge of the surveys for and construction of the cable roadway. From February, 1889, to April, 1890, Mr. Vaughan served as Assistant Engineer with the Denver (Colo.) City Cable Railway Company, in charge of surveys for and the construction of the roadway of the Larimer Street Line, and the Larimer Street and 16th Street Viaducts. He spent the remainder of 1890 in Cleveland, Ohio, with the City Cable Railway Company, in charge of the construction of one division of the road, including track and vault work.

In February, 1891, Mr. Vaughan was appointed Assistant Engineer of the Brooklyn (N. Y.) Heights Railway Company, in charge of the construction of the power-house and the installation of the plant for the Montague Street Cable Road. From July, 1891, to April, 1893, he was Assistant Engineer for the Broadway and Seventh Avenue Railway Company, New York City, in charge of the construction of a section of the cable railway from Waverly Place to 17th Street and, later, of the Houston Street Power Station, including the installation of the driving machinery.

For a short time after this, Mr. Vaughan returned to private practice as a Civil Engineer in Providence, R. I., but in April, 1895, he went to Cambridge, Mass., as Assistant Engineer in charge of the construction of several sections of the 2 500 000 000-gal. Hobbs' Brook Storage Basin of the City Water-Works; during the last six months he was Resident Engineer in entire charge of the completion of the work and of the final estimates. This engagement was followed by a short period as Assistant Engineer in charge of the surveys for a section of a high-level sewer for the Massachusetts Metropolitan Sewerage Commission.

From December, 1898, to May, 1902, Mr. Vaughan was with the Boston Elevated Railway Company as Assistant Engineer on calculations for the steel structure, in charge of the construction of pile and concrete foundations on a section of the

* Memoir prepared by Victor H. Vaughan, Esq.

road, on the design of special track work, and in charge of the re-arrangement of the tracks, installation of the third-rail, and changes in the station platforms of the Boston Subway. During the final six months of this period he was in the Road-master's Office, in charge of track repairs on the elevated structure and in the subway.

From May to September, 1902, he was with Westinghouse, Church, Kerr and Company, in charge of the surveys on a section of high-speed electric railroad in Central New York State, but while engaged at this work his health broke down, and he was obliged to resign.

For the next three years, Mr. Vaughan was with Percy M. Blake, Civil and Hydraulic Engineer, Newtonville, Mass., as Assistant Engineer in charge of surveys, designs, and estimates for water-works and water-power development and for the valuation of water-works properties in various places in Massachusetts and Connecticut.

In February, 1906, he entered the service of the Board of Water Supply, Reservoir Department, of New York City, at which time he was appointed Assistant Engineer in charge of the real estate surveys. From 1909 to 1911, Mr. Vaughan served as Section Engineer in charge of the construction of the intercepting sewer in Kingston, N. Y., and during 1912 and 1913, he occupied a similar position on bridge construction for the highway around the reservoir, including the Travers Hollow Bridge. In 1914, he was engaged on the fencing and highway surfacing contracts around the reservoir.

In April, 1915, Mr. Vaughan was transferred to the Southern Aqueduct Division at Valhalla, N. Y., where he was engaged on the final estimates for the Kensico Reservoir, including bridges and highways, until March 3d, 1916, when he retired from the Engineering Profession to spend the remainder of his life in Kingston, N. Y.

On September 21st, 1882, Mr. Vaughan was married to Miss Ida Etta Hall, of Providence, R. I. Mrs. Vaughan died at Kingston, N. Y., in the fall of 1918, after a year's sickness, and Mr. Vaughan never recovered from this blow. Worn down by the grief and sorrow of her sickness and death, in less than two years he died of pernicious anæmia, at the home of his brother, in Providence, R. I. He was laid to rest, beside his wife, in the North Burial Ground.

He is survived by a brother, Frank Leslie Vaughan, Secretary and Treasurer of the Providence Engineering Corporation, Providence, R. I., and an only son, Victor Hall Vaughan, a graduate of Harvard College, Class of 1918.

From 1906, when Mr. Vaughan joined St. John's Protestant Episcopal Church in Kingston, N. Y., he held the various offices of Vestryman, Senior Warden, and Church Treasurer. At the time of his death he was Senior Deacon of Kingston Lodge, No. 10, F. and A. M., a Past High Priest of Mount Horeb Chapter, R. A., and Commander of Rondout Commandery, No. 52, Knights Templar, of Kingston, N. Y., and a member of Cyprus Temple, Mystic Shrine, of Albany, N. Y. He was also a member of the Boston Society of Civil Engineers.

Mr. Vaughan was a man of quiet nature and fond of home life. In his profession he was a diligent and conscientious worker, never allowing his personal feelings and comfort to interfere with the carrying out of his duties and obligations. He was a wide reader, especially in the field of literature and history, and

throughout his life was much of a scholar. To those who really knew him, his extraordinary kindness and devotion to high ideals gained him a name to be envied.

"A good name is rather to be chosen than great riches, and loving favour rather than silver and gold."

Mr. Vaughan was elected a Member of the American Society of Civil Engineers, on January 31st, 1911.

CHARLES EDWARD WEBSTER, M. Am. Soc. C. E.*

DIED OCTOBER 7TH, 1920.

Charles Edward Webster was born at Mauch Chunk, Pa., on May 5th, 1842. On his paternal side he was a descendant of Noah Webster and of John Webster, former Governor of Connecticut. His paternal grandfather was a publisher in Albany, N. Y., and his father was pastor of the Presbyterian Church at Mauch Chunk, Pa., from 1837 to 1856. On his mother's side, he was of Irish descent.

Mr. Webster received his early education at private and public schools in Mauch Chunk, his native town, and was graduated from Princeton University in 1862. The year of his graduation he entered railway service, in which service he continued for various companies in many States until about 1912, when he retired.

During his long career Mr. Webster was connected with the following lines and work: From 1862 to 1868 he was Rodman and Assistant Engineer during the construction of the Lehigh and Susquehanna Railroad (Central Railroad of New Jersey); from 1868 to 1870 he was Assistant Engineer, of the Boston, Hartford and Erie Railroad, in Connecticut; in 1870 he became Assistant Engineer with the Chesapeake and Ohio Railway, and, later, Chief Engineer of the Newburg and Midland Railway; in 1871 he accepted the position of Assistant Engineer on the location of the New Jersey West Line Railroad, between Bethlehem, Pa., and Newark, N. J.; from 1871 to 1874, he was Resident Engineer of the Chesapeake and Ohio Railway, in West Virginia; in 1874, he became Chief Engineer of the Longdale Road, in Virginia; from 1875 to 1879 he acted as Resident Engineer, Division Engineer, and Assistant Chief Engineer, of the Cincinnati Southern Railway; from 1879 to 1881 he was Division Engineer of the New York and New England Railroad; from 1881 to 1883 he acted as Division Engineer and Assistant Chief Engineer of the New York, Chicago and St. Louis Railroad; in 1883 he was Principal Assistant Engineer, Rochester and Pittsburgh Railroad, Chief Engineer, Susquehanna and Allegheny Railroad, and Assistant Engineer, Boston, Hoosac Tunnel and Western Railway; from 1883 to 1885 he was Resident and Division Engineer, South Pennsylvania Railroad; from 1885 to 1886 he acted as Assistant Engineer on the Croton Aqueduct, New York City; from 1886 to 1888 he was Division and Principal Assistant Engineer, St. Louis and San Francisco Railway; in 1888 he became Chief Engineer of the Johnsonburg and Bradford Railroad; from 1889 to 1891 he acted as Chief Engineer, Schuylkill and Lehigh Valley Railroad; from 1891 to 1899 he was Assistant Chief Engineer and Chief Engineer, Lehigh Valley Railroad; from 1899 to 1900 he was Chief Engineer, Erie and Wyoming Valley Railroad; from 1901 to 1902 he acted as Division Engineer, New York Central and Hudson River Railroad; in 1902 he acted as Consulting Engineer for the Buffalo and Susquehanna Railroad; and from 1902 to 1912 he acted as Consulting Engineer for railroads and on various construction projects. Since 1912, on account of his advanced age and failing health, he had lived in retirement.

During Mr. Webster's long engineering career, he had the privilege of assisting in the planning, location and construction of many American railroads during a

* Memoir prepared by Frederick E. Schall, M. Am. Soc. C. E.

most active construction period, and through his wide experience in this work, his opinions and decisions were recognized by the Profession as well as by investors. His experience, coupled with his undaunted integrity and spirit of fairness, caused his services to be widely sought as arbiter in the settlement of disputes in construction matters.

Aside from reading the current professional and literary English publications, Mr. Webster was an ardent devotee of the French language, and up to within a few years of his death was a regular reader of French publications.

Mr. Webster was blessed with a sterling character; of an unassuming, genial and kindly nature, he was beloved, honored, and respected by his associates and co-workers and the community in which he lived. He was ever ready to lend a helping hand to the young and ambitious, to make their life easier, to give counsel and assistance toward their advancement, or to assist them in securing positions when out of employment. His unselfish spirit was prone to lead him to give more at times than his pecuniary condition warranted.

Mr. Webster was a member of the Presbyterian Church and very prominent in its various activities.

He was united in marriage on June 9th, 1870, with Mary S. Wheeler, of Southbury, Conn., who, with two children, Charles E., Jr., and Margaret V., survives him. He is also survived by two brothers and two sisters.

Mr. Webster was elected a Member of the American Society of Civil Engineers on October 4th, 1899.

PAUL LUDWIG WOLFEL, M. Am. Soc. C. E.*

DIED DECEMBER 28TH, 1920.

Paul Ludwig Wolfel was born in Dresden, Saxony, on April 19th, 1862. He was the son of Frederick E. and Emma Richter Wolfel. His father was a manufacturer, all of whose ancestors before him were business men and manufacturers. On his mother's side, the male members showed a preference for the medical profession. All his ancestors for generations back lived in Saxony.

Mr. Wolfel secured his early training in private schools and the Realschule in Dresden. He then became a student at the Polytechnikum in Dresden, from 1880 to 1885, giving special attention to bridge design under Professors Fraenkel and Mohr, and was graduated in October, 1885. He served in the German Army for one year. From 1885 to 1887, he was Assistant to Professor F. Steiner at the Polytechnikum in Prague, Bohemia, and also a Privatdocent at the same college. During this period, he was connected with the Prager Maschinenbau Actien Gesellschaft in the preparation of designs and estimates for bridges. In 1886, he was accorded a traveling scholarship in Engineering and visited Paris and London.

Mr. Wolfel came to America in June, 1888, having been recommended to the late C. C. Schneider, Past-President, Am. Soc. C. E., then Chief Engineer of the A. and P. Roberts Company (Pencoyd Iron Works) of Pencoyd, Pa., by Professor Fraenkel, who advised Mr. Schneider that he was a most promising student who had passed every branch of his studies with an average of excellence, which had not occurred previously for ten years. In 1890, Mr. Wolfel was appointed Assistant Chief Engineer of the Bridge and Construction Department, and held that position until the Pencoyd Company was merged with the American Bridge Company in 1900.

One of the policies which he put into practice during this period, and consistently adhered to in his later life, was the development of young men who came under his direction. The Apprentice Course offered in the Pencoyd Drafting Room produced many engineers who credit their success to his early guidance. Through his advice and influence many young men were also persuaded to take up a college course in engineering, who to-day hold prominent positions of responsibility and trust in the Engineering Profession. His interest in his fellow engineers revealed itself in many ways. He was one of the founders and principal supporters of the Pencoyd Club, established in 1900, principally for the use of the men employed at the Pencoyd Works.

Some of the more important structures built by the Pencoyd Iron Works, with which Mr. Wolfel was identified, were the large trainsheds, at Jersey City, N. J., and Philadelphia, Pa., for the Pennsylvania Railroad; the Manhattan Life Insurance Building in New York City, among the earlier high buildings; the Delaware River Bridge for the Pennsylvania Railroad, near Philadelphia; the Niagara Falls and Clifton Arch Bridge; and the famous Atbara River Bridge for the British Government, the order for which was received on January 24th, 1899, and which was completely erected by Pencoyd's own forces by August 19th, 1899. The late Lord Kitchener said on the completion of the contract:

* Memoir prepared by G. Lewis Taylor and W. Irving Lex, Members, Am. Soc. C. E.

"I congratulate the American foreman and workmen on the excellent success which has crowned their efforts in the erection of this bridge in the heart of Africa."

On the formation of the American Bridge Company in 1900, Mr. Wolfel became Engineer of the Railway Department, and, at the same time, Chief Engineer of the Bridge and Construction Department of the Pencoyd Iron Works and the A. and P. Roberts Company. In January, 1901, he was made Chief Engineer of the Eastern District, and in April, 1901, became Chief Engineer of the American Bridge Company.

It was a stupendous task to organize and systematize the various Engineering Departments and the methods of fabrication of the different companies entering into the newly formed larger company. This task fell principally on the Chief Engineer, and Mr. Wolfel was successful in bringing order out of chaos and establishing a high standard of efficiency. Many important bridges and buildings were built under his supervision while he was connected with the American Bridge Company.

In 1905, after a period of ill health, he became Consulting Engineer for the Company, remaining in this position until June 30th, 1908, when he resigned to assume the duties of Chief Engineer of the McClintic-Marshall Company.

An official of the American Bridge Company writes:

"Mr. Wolfel's active connection with the Pencoyd Iron Works and the American Bridge Company was ever signalized by a high type of intelligent administration. He was thorough in research, careful in analysis, honest in execution, fair-minded toward his colleagues, and evinced a brotherly interest in the men who served under him. As Consulting Engineer for the American Bridge Company, his outstanding work was done in connection with the Company's part in the reconstruction of San Francisco."

Mr. Wolfel became Chief Engineer of the McClintic-Marshall Company at the time of the construction of the Ohio River Bridge at Beaver, Pa., for the Pittsburgh and Lake Erie Railroad Company, of the New York Central Lines. This great structure was the first large cantilever to be built after the disaster to the first Quebec Bridge, and unusual care was taken in its design, fabrication, and erection. Because of its magnitude and carrying capacity, special study was given to perfecting its details of construction, in which work Mr. Wolfel's experience and clear-sighted constructive ability were invaluable. The Beaver Bridge, as a result, marked a distinct advance in bridge engineering.

In the fabrication and erection of the forty-six pairs of mitering lock-gates for the Panama Canal, the largest ever constructed, Mr. Wolfel's guidance contributed to the successful completion of the work by the McClintic-Marshall Company. At that time, this contract represented probably the largest tonnage of structural steelwork ever contracted for at one time.

Among the great bridges constructed by the Company under his supervision were the viaducts and various crossings of the New York Connecting Railroad (Hell Gate Line) on Long Island and in the Bronx; the Fratt Bridge over the Missouri River at Kansas City, and the Sciotoville Bridge of the Chesapeake and Ohio Northern Railway over the Ohio River. The erection of the Sciotoville Bridge presented problems of unusual difficulty, as it was a continuous structure

consisting of two 775-ft., riveted truss spans, and the cantilever method of erection was required. Mr. Wolfel solved the intricate problems presented in a most satisfactory manner.

He brought to the McClintic-Marshall Company organization the judgment and skill acquired by his experience, and trained the engineering members of that organization along the lines of the best practice.

He was called as a Consulting Expert by the Board of Engineers appointed to decide on the type of construction of the second Quebec Bridge over the St. Lawrence River.

Mr. Wolfel's work ever expressed his desire for the advancement of the art of steel bridge construction in excellence and stability. He enjoyed the confidence of the Engineering Profession to an unusual degree. In his dealings with other engineers, he was always fair-minded and considerate of their viewpoint. Whatever was placed in his hands was treated with care, thoroughness, and absolute integrity of purpose to achieve the best possible result. In his relations with younger men he pointed the way to a fixed aim for reliable work and a high professional standard.

Mr. Wolfel was a member of the Engineers' Society of Western Pennsylvania, American Railway Engineering Association, the American Society for Testing Materials, the Franklin Institute, the Western Society of Engineers, and the Society for the Promotion of Engineering Education. He also belonged to the Engineers' Clubs of New York and Philadelphia, the Duquesne Club of Pittsburgh, and to the Masonic Fraternity, and was a member of the German Lutheran Church. He became a citizen of the United States early in his life here.

Mr. Wolfel was married on October 11th, 1893, to Miss Emma Brecht, of Roxborough, Philadelphia, Pa., who died on August 20th, 1910. He died in Philadelphia on December 28th, 1920, and is survived by his daughter, Miss Emma Wolfel, a sister, Miss Marie Wolfel, and a brother, Egon Wolfel.

Mr. Wolfel was elected a Junior of the American Society of Civil Engineers on July 3d, 1889, an Associate Member on July 1st, 1891, and a Member on November 6th, 1895.

ANTHONY GEORGE ARMSTRONG, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 23D, 1921.

Anthony George Armstrong, the son of Anthony W. and Jessie Easton Armstrong, was born on June 20th, 1889, in Alexandria, Va. His father, a prominent attorney of Alexandria, had planned every educational advantage for his son, but he died very suddenly when Mr. Armstrong was only fourteen years of age. The sudden transition from being the supreme object of his father's care to a knowledge that he must make his own place in the world entirely unaided developed in him personal traits of independence and self-reliance out of proportion to his years. These traits predominated in his character, and although they sometimes stood in his way in connection with his work in subordinate positions, they were of inestimable value when this work grew to be of a more executive nature.

He received his education at the Episcopal High School of Virginia and was a student at the Virginia Military Institute from 1905 to 1907.

After leaving the Virginia Military Institute, Mr. Armstrong, then eighteen years old, was promised entrance to the West Point Military Academy, but through some small technicality (a matter of being ½ in. under height at the time), he lost this opportunity. In a spirit of youthful indignation, he enlisted in the United States Cavalry, at Fort Myer, Va., declaring that he would win a commission from the ranks. He was soon sent with his troop to the Philippines, where he remained for nearly three years. During this time he did some very interesting road construction, most of the time under fire from the Moros. Several of the Army officers and engineers interested themselves in him and helped him with studies which he pursued at all spare times. When his term of service was up, he decided that he would not try for an Army commission as his interest in engineering had been awakened. He was discharged from the Army in 1912, in Southern Texas, where his troop had just been ordered. He then joined the United Fruit Company, and worked, as Timekeeper and Estimator on construction for that Company, in Guatemala, until January, 1913, when he went to Panama and was employed on the Miraflores and Pedro Miguel Locks.

In the Fall of 1913, desiring to supplement his practical experience with technical knowledge, Mr. Armstrong entered the University of Virginia, where he studied until 1915. After leaving college, his work until August, 1917, was chiefly in connection with railroad engineering with such roads as the Southern and the Baltimore and Ohio Railways, on maintenance of way and construction.

From August to October, 1917, Mr. Armstrong was engaged as Civil Engineer for the Bethlehem Steel Company, at Sparrows Point, Md., in charge of material for mill construction. In November, 1917, he began work with the Curtis Bay Ordnance Depot, near Baltimore, Md., as Assistant Engineer in charge of railroad location and construction, but resigned to enlist in the Marine Corps in January, 1918. Shortly afterward he was commissioned a First Lieutenant, U. S. Marines; he was stationed almost continuously at Quantico, Va., and his work, as Orientation Officer, Assistant in charge of the triangulation survey for an artillery range, and Assistant to the Public Works Officer was of a nature in which his experience and technical knowledge were used to great advantage.

* Memoir compiled from information on file at the Headquarters of the Society.

In August, 1919, Mr. Armstrong resigned his commission in the Marine Corps and became a Field Superintendent for the Austin Company of Philadelphia, Pa., in which capacity he had charge of the construction of large industrial buildings. The nation-wide depression in construction work during the last months of 1920 affected his Company, and scarcity of contracts making it necessary for him to look for work elsewhere, he resigned this position in December, 1920. In January, 1921, Mr. Armstrong came to New York City. On January 20th, he wrote to his sister in Virginia that he had accepted the position of Consulting Engineer of the Williamsburg Light and Power Company, Incorporated, The York Laundry and Ice Company, and the Peninsula Grain Products Company, Incorporated, at Williamsburg, Va., and that he expected to visit her in a day or two *en route* to Richmond, stating, however, that he had remained in New York to attend the Annual Meeting of the Society.

On Sunday morning, January 23d, his sister received a message from New York that Mr. Armstrong had died suddenly the preceding night from gas poisoning caused by a leak in a gas tube connected with a heater in his room. It seems almost a paradox of Fate that a life so full of vigor and initiative, a mind so forceful and confident, should have been terminated by so casual an accident.

Mr. Armstrong was a member of the Kappa Alpha Fraternity (Southern). He was unmarried, and is survived by an only sister, Mrs. Jessie V. Moran, of East Falls Church, Va.

Mr. Armstrong was elected an Associate Member of the American Society of Civil Engineers on November 9th, 1920.

WILBUR VICK BANISTER, Assoc. M. Am. Soc. C. E.*

DIED JUNE 3D, 1920.

Wilbur Vick Banister was born at Pardoe, Pa., on February 6th, 1883, and there received his early education. He was graduated from the Mechanics Institute in Architecture in 1905, and spent two years at the Harlem School of Engineering, New York City.

From 1905 to 1908, Mr. Banister was engaged on the design of steel and frame buildings for the John Fulton Construction Company of New York City, and from 1908 to 1911, he was employed as Superintendent of Construction on a costly residence at Davenport Neck, Long Island, and on the Belmont Residence in New York City.

In January, 1911, he entered the employ of the W. L. Crow Company, New York City, as Superintendent on the construction of the Vanderbilt Hotel, a 22-story building of steel, concrete, and brick, costing approximately $1 000 000, which position he retained until February, 1912. He spent the next seven months as Superintendent of Construction of a 16-story loft and office building, in New York City, for the H. D. Best Company.

In October, 1912, Mr. Banister went to Vancouver, B. C., Canada, where he was engaged until February, 1915, as Superintendent of Construction, on the Canadian Pacific Railway Terminal, including a passenger station and two viaducts, costing approximately $2 000 000, for Westinghouse, Church, Kerr, and Company, of New York City.

In 1916, he entered the employ of the Stone and Webster Engineering Corporation, of Boston, Mass., and from January to August of that year, he served as Superintendent of Construction on the Dallas Interurban Terminal at Dallas, Tex., and from August to January, 1917, he was engaged in a like capacity on the construction of a reinforced concrete and brick factory building for the B. F. Goodrich Company, at Akron, Ohio.

In January, 1917, Mr. Banister entered the Boston office of the Stone and Webster Corporation, and later in the same year, he was made Superintendent for that Corporation on the construction of the Administration Building for the Whitins Machine Company, at Whitinsville, Mass. In 1918, he was sent to Dover, N. J., as Superintendent on the construction of the Picatinny Arsenal, and, in 1919, he went to California as Advance Agent for the Corporation.

In November, 1919, Mr. Banister returned East and entered the employ of the Thompson-Starrett Company of New York City. He was appointed Superintendent on the construction of the Ambassador Hotel Annex, Ritz-Carlton Hotel, and the Convention Hall, at Atlantic City, N. J., where he remained until his death which occurred at his home there on June 3d, 1920, after an illness of four years.

He was married on April 1st, 1916, to Miss Myrtle A. McClean, of Long Beach, Cal., who survives him.

Having had to work his own way to obtain his education, Mr. Banister became an ardent student and a great reader, and his wide knowledge of engineering was

* Memoir compiled from information on file at the Headquarters of the Society.

obtained through painstaking study, experience, and contact with men of higher learning. He had a keen analytical mind, and although cut down at the beginning of a career of great promise, he left no small monument in good work well done in his chosen profession.

Quiet and unassuming in manner, possessed of an unselfish disposition, a pleasing personality, with a strong sense of fairness and justice toward all with whom he came in contact, and of scrupulous integrity, Mr. Banister will be greatly missed by a wide circle of friends.

He was a member of Manitou Lodge No. 106, F. and A. M., of New York City, and of the Chelsea Baptist Church, of Atlantic City, N. J.

Mr. Banister was elected an Associate Member of the American Society of Civil Engineers on June 11th, 1917.

HARRIS DANIEL BUCKWALTER, Assoc. M. Am. Soc. C. E.*

DIED MAY 11TH, 1918.

Harris Daniel Buckwalter, the son of W. Howard and Anna C. Buckwalter, was born at Downingtown, Pa., on November 21st, 1887. In 1888, his parents moved to Royersford, Pa., where the boy grew to manhood. He was educated in the public schools of Royersford and was graduated from the High School in 1906. He studied Civil Engineering at Pennsylvania State College, from which he was graduated with the Class of 1910, with the degree of B. S. in Civil Engineering. He received the degree of C. E. in 1916.

During his college vacations, Mr. Buckwalter was employed as a Rodman for the Harris Engineering Company, Chainman and Leveler for the Pennsylvania State Highway Department, and with Mr. J. P. Harding, on the design and drafting of steel construction. After his graduation, in August, 1910, he entered the employ of the Indiana Bridge Company, at Muncie, Ind., where he was engaged on the designing and detailing of steel bridges, mill buildings, etc.

In December, 1910, Mr. Buckwalter was appointed Assistant City Engineer of Muncie, Ind., in charge of surveys for sewers, streets, and parks, until July, 1911. when he went to the New York Holding and Construction Company as Construction Engineer on the design and erection of walls, floors, and roofs for garages, residences, and hotels, using interlocking tile and concrete.

From November, 1911, to December, 1914, he was with the Pennsylvania State Highway Department, until August, 1912, as Chief of Party on topographic surveys, relocation of roads, etc., and, afterward, as Designer on steel and concrete highway bridges, viaducts, etc.

On December 1st, 1914, Mr. Buckwalter was transferred to the Bureau of Engineering of the Public Service Commission of Pennsylvania as Assistant Engineer in charge of grade-crossing elimination, etc., and remained in this position until the United States entered the World War, when he volunteered his services to the Government on March 31st, 1917.

On June 19th, 1917, he was commissioned a Captain of Engineers, United States Reserve Corps, and was called to active service on September 5th, 1917. He was sent for training to American University, Washington, D. C., where he remained until December 10th, 1917, when he was ordered to Camp Gordon, Atlanta, Ga., for special duty with the 307th Engineers.

On January 27th, 1918, Captain Buckwalter sailed for France and on his arrival was assigned to Company A of the 1st Engineers, U. S. A., which, at that time, was at the front in the Toul Sector. On April 6th, 1918, Captain Buckwalter was appointed Adjutant of the Regiment and served as such until May 8th, 1918.

On the night of May 11th, 1918, as Engineer Officer in command of a company of infantry which was engaged in the construction of trenches on the Belle Assise Farm, about a mile east of Broyes, France, he was instantly killed by a high-explosive shell of the enemy, which landed near him on the piked road. He was buried by the 1st U. S. Engineers with military honors, in the U. S. Military Cemetery at Broyes. On March 26th, 1921, his body was received from overseas

* Memoir prepared from information on file at the Headquarters of the Society.

and re-interred in Fernwood Cemetery at Royersford, Pa. He is survived by his parents and one sister.

Captain Buckwalter was a young man of great promise, clean, ambitious, conscientious, with a strong sense of duty and loyalty, of excellent judgment and a clear and orderly way of thinking. Although only in his thirty-first year at the time of his death, he had attained high rank in his profession, and had held, for a number of years, positions of such responsibility as seldom fall to so young a man. He also had that happy faculty of making friends wherever he went and of retaining them; he was always hopeful and cheerful, laying his plans for the future and ever ready to help others.

On being advised of the death of Captain Buckwalter, the following resolutions were adopted by the Chief and his fellow members of the staff of the Bureau of Engineering of the Public Service Commission of Pennsylvania:

"Whereas, Harris D. Buckwalter was for several years in the employ of the Commonwealth of Pennsylvania as Bridge Engineer in the Bureau of Engineering of the Public Service Commission, and was an earnest, faithful and efficient State officer, and while in said employment was commissioned Captain in the Engineer Officers Reserve Corps of the United States Army, and

"Whereas, Harris D. Buckwalter with six other members of the staff of the Bureau of Engineering, all commissioned officers, entered the Federal service with a full realization of the sacrifice which said service might entail, and

"Whereas, Captain Harris D. Buckwalter is the first commissioned officer in the employ of the Commonwealth, so far as we are informed, to make the full sacrifice, in the pending crisis, in patriotic support of democracy and his country's institutions;

"Therefore, Be it Resolved, That in his heroic death, Captain Harris D. Buckwalter has accomplished his life work and achieved a star of glory whose light will continue to inspire others to acts of bravery and sacrifice to the end that Liberty, Peace on Earth and Goodwill Towards Man shall prevail, and

"Be it Further Resolved, That this resolution be framed and hung in the office of the Bureau of Engineering and a copy forwarded to the parents of the deceased."

The City of Harrisburg, Pa., through its Park Department, has planted memorial trees in honor of the soldiers and sailors of the city who died in war service, and Captain Buckwalter's name appears on the list of those so honored.

Captain Buckwalter was widely known among Penn State Alumni through his leadership in the alumni activities of the Harrisburg Penn State Association, of which organization he was Secretary-Treasurer for several years. It was largely due to "Buck" that the Harrisburg Club became one of the strongest and most active of the local Alumni clubs.

He was a member of the Engineers Club of Harrisburg, and also a member of Royersford Lodge, No. 585, F. and A. M.

Captain Buckwalter was elected an Associate Member of the American Society of Civil Engineers on May 31st, 1916.

ROBERT DELANO CUSHING, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 23D, 1903.

Robert Delano Cushing, the son of George A. and Lucia (Mitchell) Cushing, was born on August 16th, 1868, in New York City, where his father, who was a civil engineer, was connected for many years with the Park Department and the Croton Water Supply work. On his mother's side, Mr. Cushing was descended from Pilgrim stock, while his paternal ancestors had been long resident in Lunenburg, Mass. His education was received in private and public schools, and he prepared for college at the Bridgewater, Mass., High School. From 1887 to 1891, he studied at the Massachusetts Institute of Technology, specializing in the Mechanical and Electrical Courses.

From September, 1891, to September, 1892, Mr. Cushing was employed in the Mechanical Department of the American Bell Telephone Company, at Boston, Mass. From the latter date until September, 1898, he was connected with the Engineering Department of the Fitchburg Railroad, at Fitchburg, Mass., as an Assistant Engineer in responsible charge of various kinds of railroad construction and maintenance.

From September, 1898, to June, 1899, Mr. Cushing was employed by the Boston Elevated Railway Company, at Boston, as an Assistant Engineer in charge of a Division Office in connection with the building of the elevated lines.

In 1899, his health began to fail, and soon thereafter he went to Lunenburg, Mass., where his mother then resided, remaining there until his death on August 23d, 1903.

Mr. Cushing was unmarried and was survived by his mother, a sister, and two brothers.

He was a delightful companion, of considerable musical ability, and was resourceful and ingenious in all his undertakings.

Mr. Cushing was elected an Associate Member of the American Society of Civil Engineers on September 6th, 1899.

* Memoir prepared by H. W. Hayes, M. Am. Soc. C. E.

CHARLES CLAYTON HUFF, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 1ST, 1920.

Charles Clayton Huff, a son of James Clayton and Virginia (Sawyer) Huff, was born at Atlanta, Ga., on April 22d, 1882. After completing the Grammar and High School courses in Atlanta, he studied at the Georgia Institute of Technology, from which he was graduated in 1903, with the degree of Bachelor of Science in Mechanical Engineering. A remarkable record in attendance was made by Mr. Huff in that he was not once absent or tardy from the time of entering school until his graduation.

The first seven months of Mr. Huff's professional work were spent in the employ of the Atlanta Terminal Company, as Draftsman. Until September, 1904, he served as Architectural Draftsman and Inspector for Messrs. Frank Lockwood, of Montgomery, Ala., and T. F. Lockwood, of Columbus, Ga. The West then called Mr. Huff to Los Angeles, Cal., where he began a service of more than eight years with the Los Angeles and Salt Lake Railroad Company, as Draftsman in the office of the Division Engineer of Maintenance of Way. During the last few years of his connection with this company he served as Chief Draftsman and Office Engineer.

From January, 1913, to March, 1915, Mr. Huff was Junior Engineer Draftsman in the office of the U. S. District Engineer at Los Angeles, working especially on river and harbor, irrigation, and flood-control work. In March, 1915, he was promoted to be Junior Engineer in the same office in charge of the design of, and estimates for, 14-in. rifle batteries, 12-in. mortar batteries, fire-control installations, and mine building, all of this work being for Fort McArthur, Los Angeles Harbor. From October, 1917, until his death, Mr. Huff was at Fort Rosecrans, San Diego, Cal., in charge of the U. S. Engineer Field Office, his work covering the construction of fortification works, including the installation of the fire-control system for new mortar and rifle batteries, and the modernizing of existing batteries. In this connection, he also constructed water, road, and sewerage systems.

Mr. Huff's death occurred on August 1st, 1920, at Fort Rosecrans, California, after a severe illness of many months. Burial took place in Inglewood Cemetery, Los Angeles, Cal.

Mr. Huff had no particular hobby or interest, his professional work being the great and compelling interest in his life. He was a deep student and a keen thinker, and gave great promise of advance to a high position among his fellow professional men. He was a member of the Sigma Nu Fraternity, Royal Arcanum, Masonic Lodge, and American Association of Engineers.

On March 15th, 1906, he was married to Miss Vivian Bell Curtis of Los Angeles, Cal., who, with two children, Vivian Virginia and James Curtis Huff, now living in Los Angeles, survives him.

Mr. Huff was elected an Associate Member of the American Society of Civil Engineers on November 12th, 1913.

* Memoir prepared by A. F. Barnard, Assoc. M. Am. Soc. C. E.

JULIUS JAMES KNOCH, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 27TH, 1920.

Julius James Knoch, the son of Herman and Amelia (Roebling) Knoch, was born at Saxonburg, Pa., on January 12th, 1863. He was one of six children, five of whom were boys. His parents were of sturdy Pennsylvania stock, his mother being a niece of W. A. Roebling, the builder of the Brooklyn Bridge.

Mr. Knoch's early education was obtained in the public schools near his home, after which he entered Grove City College, Grove City, Pa., in 1882. From this institution he received the degree of B. S. in 1886 and that of M. S. in 1888. He entered Cornell University the same year and received the degree of C. E. in 1892.

Mr. Knoch was Instructor in German and Mathematics at Grove City College from 1886 to 1888. While a student at Cornell University and after his graduation he was Assistant Engineer to the late Professor C. L. Crandall, M. Am. Soc. C. E., his work including a 900-ft. steel bridge over the Susquehanna River, at Oswego, N. Y.

From June, 1893, to 1894, he was Adjunct Professor of Civil Engineering at the University of Arkansas, at Fayetteville, Ark., and Associate Professor from 1894 to 1897. In 1897, he was made full Professor in charge, which position he held continuously until July 1st, 1919. In addition to his other duties, he acted for many years as Superintendent of Buildings and Grounds of the University.

In 1898, Professor Knoch was made City Engineer of Fayetteville, Ark., and fulfilled the duties of that office until 1919. For a number of years he was also senior member of the firm of Knoch and Knott, and while thus associated was interested in the design and supervision of many important highway bridges and public works in Washington County, Arkansas.

Professor Knoch resigned the Chair of Civil Engineering at the University of Arkansas in June, 1919, to enter the firm of Carter and Knoch, at Little Rock, Ark., a consulting engineering partnership which continued until his death. As a member of this firm, he was interested in highway improvements in various parts of Arkansas, many of the projects being of great magnitude.

His success as a teacher of engineering was phenomenal. When he took charge, the Civil Engineering Department of the University of Arkansas was a branch of the Mechanical Engineering Department, but within a few years a separate College of Civil Engineering was created, with Professor Knoch at its head. The growth of this Department was rapid, and continued until his resignation. It became known as one of the best civil engineering schools in the Southwest, and numbers its successful graduates and students by the thousands, much of its reputation being due to the ability and to the tireless efforts of Professor Knoch during his twenty-six years of service.

While at the University Professor Knoch was one of the most popular members of the Faculty. A man of great mental ability and keen wit, he was the first to make or appreciate a joke in or out of the classroom. He was of a retiring dis-

* Memoir prepared by V. P. Knott, State Highway Engr., Little Rock, Ark.

position, unassuming, yet the best of companions. His office was always open to students, whether for business or pleasure, and his home was numbered among the most hospitable of the college town. It was the custom of Professor and Mrs. Knoch to entertain frequently the Junior and Senior Civil Engineers. On their visits to their Alma Mater, the old "grads" were always welcome guests, while those far away carried on as regular a correspondence with Professor Knoch as with members of their own families, showing the character of the man and accounting for his success and for the love and esteem in which he was universally held by his students.

Professor Knoch was married in May, 1893, to Miss Amelia Keeler, of Ithaca, N. Y. Of this union two sons, Elmo and Lester, were born.

He was a member of the Society for the Promotion of Engineering Education and the Arkansas Engineering Society.

Professor Knoch was elected an Associate Member of the American Society of Civil Engineers on October 2d, 1901.

RALPH BARTON MANTER, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 2D, 1911.

Ralph Barton Manter was born in Milo, Me., on August 29th, 1873. He was graduated with the degree of B. C. E. from Maine State College in 1896, after which he entered the service of the Metropolitan Water Board of Boston, Mass., filling the positions of Rodman, Instrumentman, and Inspector, respectively, until 1900.

In March, 1900, Mr. Manter accepted employment as Assistant Engineer with the Isthmian Canal Commission for duty in Nicaragua, where he remained until the completion of the investigations in connection with the canal route across the Isthmus.

In July, 1901, he was appointed Provincial Supervisor of the Province of Zambales, Philippine Islands, in charge of all public works in that Province. Mr. Manter retained this position until November, 1902, when he accepted an appointment as Resident Engineer with the Canton-Hankow Railway in China, where he was engaged on the construction of stations, bridges, etc.

When active work was begun by the United States Government on the Panama Canal, Mr. Manter again, in the spring of 1904, entered the employ of the Isthmian Canal Commission. He remained on that work through the early years and proved himself a valuable Assistant during the preliminary investigations.

After the preparatory work on the Canal had been completed, however, he left the employ of the Isthmian Canal Commission and went to Colombia, and, later, to Brazil, where he entered the service of the Madeira-Mamore Railway Company. While engaged on preliminary survey work at one of the outpost camps, on the banks of the Madeira River, a tributary of the Amazon, Mr. Manter was taken ill and died on February 2d, 1911. He was laid to rest, with all the final honors which his comrades could bestow, on a gentle slope overlooking the river and the railway to the construction of which he had contributed his last services.

Through his early death the Engineering Profession lost a valuable member. His natural ability, combined with his technical training, insured a promising future, and his integrity and loyalty endeared him to his comrades and fellow colleagues. The writer who was associated with him in various fields during his career, and his associates on the work in Brazil at the time of his death, felt his loss deeply.

Mr. Manter was elected an Associate Member of the American Society of Civil Engineers on June 15th, 1904.

* Memoir prepared by H. F. Dose, M. Am. Soc. C. E.

FREDERIC CRAMER MIEROW, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 18TH, 1921.

Frederic Cramer Mierow was born in Brooklyn, N. Y., on May 26th, 1888. Following his school days at the Lakewood, N. J., High School and the Pingry School in Elizabeth, N. J., he entered Princeton University with the Class of 1910 and, later, continued his engineering course at the University of South Carolina.

After leaving college, Mr. Mierow was employed on a number of large engineering projects in various parts of the United States, namely, the electrification of the Hoosac Tunnel; power-house and high-power line construction at Mauch Chunk, Pa., for the Lehigh Coal and Navigation Company; and the project for the control of the flood-waters of the White-Stuck and Puyallup Rivers at Puyallup, Wash., for the Inter-County River Improvement. For the past few years, he had been connected with the New Jersey State Highway Commission as a Resident Engineer on road construction. During the World War, Mr. Mierow tried to enlist in an Engineering Corps, but was rejected on account of the condition of his heart.

He died very suddenly on January 18th, 1921, at his home in Lakewood, N. J., after an illness of only five days. He had been suffering from a throat ailment which proved too great a strain on his heart. He is survived by his parents, Mr. and Mrs. Charles Mierow, of Lakewood, N. J., a widow, and a baby girl of three months, and also by two brothers, Charles C. and Herbert E. Mierow, of Colorado Springs, Colo.

W. J. Roberts, M. Am. Soc. C. E., under whom Mr. Mierow was formerly employed, writes of him as follows:

"Mr. Mierow was a very satisfactory assistant, conscientious in his work, ambitious to learn, always doing his part well. He was a young man of excellent character and very dependable. His associates speak very highly of his genial ways, and report him to have been a very likable fellow."

He was a member of the Princeton Engineering Association and the American Association of Engineers.

Mr. Mierow was elected an Associate Member of the American Society of Civil Engineers on November 9th, 1920.

* Memoir prepared from information on file at the Headquarters of the Society.

WALTER SCOTT OBERMEYER, Assoc. M. Am. Soc. C. E.*

DIED JULY 19TH, 1918.

Walter Scott Obermeyer was born on June 13th, 1888, at Chicago, Ill., where he lived until he was six years old. His parents then moved to Lansing, Kans., where they lived for four years, and then to Huntington, Ind., which they left in 1903, to come to Pittsburgh, Pa.

Mr. Obermeyer's early education was thus obtained in several places, and, later, he secured his technical training while at work by attending night classes at the Carnegie Institute of Technology for three years. He also took a course for one year at the University of Pittsburgh.

Mr. Obermeyer began his professional work as a Draftsman with the Bureau of Filtration, Pittsburgh, Pa., in 1905; in 1910 he was transferred to the Bureau of Water. In 1911, he changed to the office of Morris Knowles, M. Am. Soc. C. E., Consulting Engineer, where he remained until his death, except for a short period in 1916, when he was with J. Toner Barr, M. Am. Soc. C. E.

His experience covered both construction and design, with a predominance of the latter, largely on water-works systems. He was attentive to his work, studious, and desirous of progressing in his profession. Mr. Obermeyer was a member of St. John's Lodge No. 219, F. and A. M., of Pittsburgh.

He is survived by his father, mother, one sister, Lola (Mrs. G. W. Kirkenbush), and one brother, William H. Obermeyer.

Mr. Obermeyer was elected a Junior of the American Society of Civil Engineers on April 7th, 1911, and an Associate Member on April 17th, 1917.

* Memoir prepared by Morris Knowles, M. Am. Soc. C. E.

AMBROSE PACKARD, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 22D, 1919.

Ambrose Packard, the son of Josiah Smith Packard, of Enfield, Mass., and Margaret E. (Woodruff) Packard, of St. Davids, Ont., Canada, was born in Niagara Falls, N. Y., on March 2d, 1870. He came of a long line of New England ancestry, being a direct descendant of Samuel Packard, of Windham, England, who came with his wife and child to this country in 1638 on the ship, *Diligence*, and first settled at Hingham, Mass.

With the exception of a few years, Mr. Packard was a resident of Providence, R. I., which was his home at the time of his death. He was educated in the public schools there, and after two and one-half years at the Massachusetts Institute of Technology he became actively engaged with his father in the dredging business.

On the death of his father, in 1911, Mr. Packard became the active head of the business and at the time of his death, operating under the name of the J. S. Packard Dredging Company and Packard Hydraulic Dredging Company, Ambrose Packard was the biggest and best known dredging contractor in New England. To describe or even enumerate the various works he carried through to a successful conclusion would exceed the limits of this memoir.

Success in business is a goal to many, but with Mr. Packard it was merely a means to an end. An idealist, a poet, a humanitarian, success with him meant ability to help others. One might think such qualities, however admirable, were not adapted to aid in the progress of business—many would declare them fatal—but Mr. Packard proved that such qualities can exist (not merely in a superficial way, to be dragged to the surface for exhibition in polite circles), but as the living, ever-present fundamentals of a man's character and go hand in hand with every-day affairs. A man of the most retiring disposition, he built up a large and profitable business in the face of keen competition. Absolutely scrupulous himself, he met and disarmed unscrupulousness in others. A man of deep religious feelings, exemplary habits, and simple tastes, he was most liberal in his judgment of those leading less careful lives. In him survived the sterling merits of his New England ancestry, but tempered with a kindly cheerfulness that rendered his society a pleasure and privilege to all who knew him.

Attentive to his business and devoted to his family, Mr. Packard yet found time for innumerable charities that were known to few beyond the recipients—not the careless charity of merely giving money, but the real charity of personal service. He could help without preaching, and none was so poor or degraded as to fall beyond the scope of his humanity. Attended, as such efforts must be, by many disappointments and apparent failures, they never served to embitter his spirit nor to check the quiet cheerful energy of his efforts.

In the absorbing struggle of every-day business the news of the death of a fellow worker, and quite possible competitor, is usually received with indifference or a quickly passing regret. Perhaps no greater eulogy could be pronounced on the life

* Memoir prepared by W. V. Polleys, Esq., Providence, R. I.

and character of Ambrose Packard than was expressed in the grief and shock with which news of his death was received by his business associates and employees.

In February, 1902, he was married to Helen, daughter of William Morris and Janet P. (Currie) Imbrie, of New York City, who, with two sons, survives him. In addition to his immediate family, Mr. Packard is survived by his mother, a brother, and four sisters.

Mr. Packard was elected an Associate Member of the American Society of Civil Engineers on February 4th, 1913.

JAMES HILTON SHERMAN, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 2D, 1920.

James Hilton Sherman was born on April 7th, 1884, in Detroit, Mich. He received his early education in the city schools of Detroit, and his engineering training at Cooper Union, in New York City, and from various private tutors. Being the oldest boy of a large family of only moderate means, he was thrown on his own resources early in life, and his success in his chosen profession was largely due to his own persistent efforts and splendid initiative.

During his summer vacations in 1903 and 1904, Mr. Sherman was given employment by the Pere Marquette Railroad, at Detroit, as Rodman, Chainman, and Draftsman. In September, 1905, he secured a position with the New York Central and Hudson River Railroad, in New York City, as Draftsman and Instrumentman, his work being mostly in connection with reinforced concrete retaining walls, which attracted him to a thorough study of reinforced concrete design.

In March, 1907, Mr. Sherman returned to Detroit to accept a position with the Truscon Steel Company. He worked his way up with this company from Draftsman to Designer, Estimator, and Salesman and, in 1908, was assigned to the District Office in Kansas City, Mo. In September, 1912, he resigned to become Manager of the Metal Lath Department of the American Rolling Mill Company, at Middletown, Ohio. He remained with this company until October, 1913, when he returned to Kansas City as Assistant District Sales Manager for the Truscon Steel Company.

Realizing the great field for reinforced concrete grain elevators and flour mills, Mr. Sherman prepared himself especially for that class of work and in January, 1915, he became a member of the Lehrack Contracting and Engineering Company, of Kansas City, serving in the capacity of Manager and Chief Engineer. In 1917, he assisted in the organization of the Reyburn and Sherman Construction Company of Kansas City. In January, 1918, he accepted a position as Chief Engineer for the Burrell Engineering and Construction Company, of Chicago, Ill., and remained with that company until war conditions caused a cessation of construction work.

Having a desire to enter the service of his country in some capacity, and not being accepted in the Officers' Training Division, Mr. Sherman went to Camp Pike, Arkansas, to assist as a civilian engineer in the construction of the camp, remaining there until after the Armistice was signed.

He then returned to Kansas City and organized the Sherman Engineering Company of which he was President at the time of his death. This company specializes in the design of grain elevators, flour mills, and kindred structures. Due to the efforts of Mr. Sherman and his chosen associates, it has made a remarkable growth during its short and very successful life.

Mr. Sherman's death was due to an operation which was not expected to be of a serious nature, and came as a great shock to his many friends. He is survived

* Memoir prepared by Roy M. Wyatt, Esq.

by his wife, who was Miss Jessie Hunt, of Kansas City, his mother, Mrs. George Sherman, of Detroit, Mich., and five brothers and three sisters.

Mr. Sherman was one of the best types of self-made engineers. His character was of the highest, and his dealings with men above and below him were always featured by his fine courtesy and uprightness. Having been the oldest son, he was the "Big Brother" to his family, and his personality was such that to know him at all was to know him as a friend and brother.

He was a member and ardent worker of the First Baptist Church, the City Club, the Meadow Lake Golf Club, and the Chamber of Commerce, all of Kansas City, Mo.

Mr. Sherman was elected an Associate Member of the American Society of Civil Engineers on May 15th, 1917.

BURT STIMSON, Assoc. M. Am. Soc. C. E.*

DIED MAY 26TH, 1919.

Burt Stimson was born at Kalamazoo, Mich., on April 20th, 1863. He attended the public schools of that city, having been graduated from the High School, and also spent two years at the Baptist College there. Having been one of seven children, Mr. Stimson had to leave college without finishing the course, in order to go to work and assist in the care of the family. His father was a civil engineer, having been employed as a Locating Engineer with the Union Pacific Railroad Company for a number of years, and Mr. Stimson and three of his brothers followed the same profession.

The family having moved to Council Bluffs, Iowa, which became the permanent home of his parents, Mr. Stimson entered the service of the Union Pacific Railroad Company, in 1883, and was engaged with field parties on location work. This work took him to Utah, Montana, Wyoming, Colorado, etc., until 1890, when he was placed in charge of three residencies, one after another, on the Oregon Short Line and Utah Northern Railway, under the Division Engineer, J. Q. Barlow, M. Am. Soc. C. E.

In 1891, Mr. Stimson returned to Michigan to take charge of construction on the Chicago and West Michigan Railroad, at Barker Creek, Mich., where he remained until February, 1892. This work consisted of railroad construction through swampy land and, although very heavy, differed greatly from that with which he had been connected in the mountains of the West.

From January to May, 1893, Mr. Stimson had charge of a locating party in Sacramento Valley, in Arizona, for the White Hills Mining Company.

After the severe business panic of 1893, when so many engineers were without employment, he assisted in building up a bankrupt drug store in Denver, Colo. Later, for a year and a half, he did the same thing for a depleted grocery business, thus displaying versatility and a determination to be always busy. He was afterward employed on two short Colorado railroads, from Florence to Cripple Creek and from Boulder to Ward, respectively, during the whole time of their construction.

From February to June, 1899, he was in charge of 20 miles of construction on the El Paso and North Eastern Railroad, under the Chief Engineer, H. A. Sumner, M. Am. Soc. C. E., and from June, 1899, to April, 1900, he was engaged on maintenance of way on the Union Pacific Railroad, under William Ashton, Division Engineer.

Mr. Stimson then went to Utah and until August, 1900, was engaged on the running of a location line, in charge of the party, for a railroad from Kelton north to the copper mining country.

From September, 1900, to March, 1902, he held the position of Roadmaster on the Colorado and Southern Railroad, and from September, 1902, to January, 1903, was employed as Transitman on location work for the Denver, Northwestern and Pacific Railroad (Moffat Road). He was then given charge of

* Memoir prepared from information on file at the Headquarters of the Society.

the construction on Division No. 3 of this line, which construction consisted of 4½ miles of heavy mountain work, including eleven tunnels, and was finished in August, 1904.

From September, 1904, to February, 1906, Mr. Stimson was employed by the Colorado Coal and Coke Company, of Denver, Colo., on surveys of its coal properties and mines.

In February, 1906, he entered the service of the Western Pacific Railroad Company as District Engineer in charge of 20 miles of heavy construction work in Feather River Canyon, but resigned in September of the same year to accept a position as Division Engineer on the Montana Railroad (part of the Chicago, Milwaukee, and St. Paul System), with headquarters at Lombard, Mont. In this capacity Mr. Stimson had charge of 40 miles of construction from Lombard to Ringling, Mont. He remained in this position until August, 1907.

From August, 1907, to August, 1909, he had charge of several short surveys and of some construction work in Oregon. He also did some street paving and some city sewer work in Portland, Ore.

In August, 1909, Mr. Stimson entered the employ of the Oregon Trunk Railway and was placed in charge of a locating party on a line up the Dechutes River Canyon, in Oregon, under G. A. Kyle, M. Am. Soc. C. E., Chief Engineer. Of this work, Mr. Kyle has written, as follows:

"This was a very important line, on account of the fact that we had to raise it in several places from 50 to 150 ft. above water level to accommodate several dams that the U. S. Government proposed to build, which threw the line into very heavy construction and required very delicate work on the part of the Locating Engineer to balance the quantities properly and place the line so that it could be most economically built and operated. Mr. Stimson, I am pleased to say, performed the work satisfactorily to myself and the Railway Company, of which John Stevens, M. Am. Soc. C. E., was President."

From January to May, 1910, Mr. Stimson was employed as Location Engineer on the North Coast Railroad (part of the Oregon-Washington Railway System), on the examination of surveys, and, from May to October, 1910, he was in charge of location on the line of the Oregon Electric Railroad.

In October, 1910, he went to Spokane, Wash., and was engaged on work in the City Engineering Department until May, 1912, when he returned to the Oregon-Washington Railway Company on the examination of surveys in the Cascade Mountains.

In March, 1913, Mr. Stimson re-entered the service of the Chicago, Milwaukee and St. Paul Railway Company, as Locating Engineer in charge of the work on the Puget Sound and Wallape Harbor Railway until September, 1913. In 1914 and 1915, he was engaged on work for the Valuation Department of the Company, and in 1916 and 1917, served as Division Engineer in charge of the construction of the Big Blackfoot Extension from Big Blackfoot Junction to Clearwater, Mont. The revision of the original location of this work, suggested and made by Mr. Stimson, with the approval of the management, resulted in a great saving in the cost of construction, as well as betterment of the line.

In 1917, Mr. Stimson was transferred to Chicago, Ill., and was engaged again in the Valuation Department of the Company until his death, which occurred on May 26th, 1919. He had been working on the Sioux City and Dakota Division

when he was taken ill with intestinal trouble. He was on his way home, with Mrs. Stimson, when he suddenly became worse and died as the train was nearing Chicago.

Mr. Stimson was married in 1892 to Miss Edith Maynard, of Council Bluffs, Iowa, who, with two daughters and a son, survives him.

He was a very skilful location engineer, especially on heavy mountain work, and was also highly competent on construction. A man of the highest integrity, he was possessed of a disposition and personality that made and kept for him many friends. He was especially devoted to his family.

In his early manhood, Mr. Stimson had attended the Congregational Church, but afterward he had united with the Unitarian Church, in Portland, Ore. It is said of him that "he was a practical Christian and found his highest satisfaction in doing good for others, having helped many of his friends and associates through trials of sickness and death and having given strength and comfort to those in need."

He had been, for eighteen years, a member of the Modern Woodmen of America and had taken the first degree in the Masonic Order.

Mr. Stimson was elected an Associate Member of the American Society of Civil Engineers on September 12th, 1916.

GEORGE WELLMAN PARSONS, Assoc. Am. Soc. C. E.*

DIED AUGUST 15TH, 1919.

George Wellman Parsons was born at Bennington, Vt., on August 16th, 1841. He received his education at the Ball Academy in Bennington, and shortly after his graduation, about 1859, his family moved to Harrisburg, Pa., where he became connected with the Harrisburg National Bank and remained for several years. He then became associated with his father in the manufacture of agricultural implements at Harrisburg.

In 1874, Mr. Parsons entered the service of the Pennsylvania Steel Company as Assistant to the Superintendent, Maj. Luther S. Bent. He became Acting Superintendent during Maj. Bent's prolonged illness, and on the latter's recovery was made Assistant Superintendent. He served in this capacity for about six years and was then assigned to the position which proved to be his life work, that of Superintendent of the Frog, Switch and Signal Department. This Department had been established only a short time before and was still in an undeveloped condition, but Mr. Parsons' untiring industry soon developed it into one of the leading departments of the plant. Considerable progress was made in the development of signals which were the predecessors of the block signal system of to-day.

The result of this work was sold to the Union Switch and Signal Company, in 1891, in order to concentrate the energies of the Department on frogs, switches, crossings, and switch stands. In the following years Mr. Parsons invented and patented many improvements in frogs, switches and switch stands and the business developed into considerable volume under his guidance. Among the most notable devices were the "Anvil Face" frog, the forerunner of present-day first-class construction, and the improvements embodied in the "New Century" switch stand which is the standard of the United States Army, adopted for and during the World War.

In 1899, the product of the Department had reached such volume that the possibilities of expansion on the existing site had been exhausted and a new and modern plant was built under Mr. Parson's direction and supervision. It includes 10 acres under roof and involved the expenditure of more than $1 000 000.

In 1904, Mr. Parsons was given a well earned relief from active duty as Superintendent, and after a European tour he was placed in charge of patent matters for the Company, in which capacity he served until his death.

Mr. Parsons was married in 1864 to Margaret Gilliard, of Harrisburg, and she and their six children, James G., Charles R., Hial K., Jessica, Harriet, and Marguerite, survive him.

Mr. Parsons' outstanding characteristics were industry and thoroughness. The modern short-hour day had no attraction for him, and the midnight oil was liberally used. His judgment was highly esteemed by those engineers with whom he came in contact, and rarely proved at fault.

Mr. Parsons was a Charter Member and the second President of the Engineers' Society of Central Pennsylvania, in 1905. He was an active member of the Dauphin County Historical Society, and a member and Warden of Trinity Protestant Episcopal Church of Steelton, Pa.

Mr. Parsons was elected an Associate of the American Society of Civil Engineers on September 3d, 1884.

* Memoir prepared by C. A. Alden, M. Am. Soc. C. E.

HAROLD LA SALLE FISKE, Jun. Am. Soc. C. E.*

DIED OCTOBER 6TH, 1918.

Harold La Salle Fiske was born in Beatrice, Nebr., on December 13th, 1888. In 1900, his parents moved to Lincoln, Nebr., where the boy entered the public schools. He was graduated from the High School of that city when he was 14 years old.

The year following he was employed in the branch office of the Armour Company, in Lincoln, and then entered the University of Nebraska. After spending three years at the University, Mr. Fiske felt that he wanted some practical work in his chosen profession, Civil Engineering. He, therefore, went to Idaho, where he was engaged for a year on irrigation work. He then returned to the University of Nebraska to finish his course and was graduated with honors in 1910 with the degree of B. Sc. in C. E. Mr. Fiske made a very enviable record as a student at the University, having been connected with various student activities as well as a member of the Sigma Tau Fraternity.

After his graduation in 1910, he entered the employ of the Government on the Panama Canal and from August, 1910, to January, 1912, served as Transitman and Junior Engineer in charge of the construction of a 7-mile division of the relocated line of the Panama Railroad, remaining on this work until trains were running over the line. He also served from January to July, 1912, as Transitman for the Isthmian Canal Commission.

On his return to the United States, Mr. Fiske entered the employ of the Foundation Company of New York City in October, 1912, and until May, 1913, had charge, as Night Engineer, of the construction of the Western Union Building in New York City. He was then engaged as Engineer on the construction of Lock No. 8, Dam No. 4, of the New York State Barge Canal, at Scotia, N. Y., in the sinking of pneumatic caissons, and the construction of lock, guide walls, and piers. Both these undertakings demanded a high class of engineering skill and ability.

After leaving the Foundation Company, Mr. Fiske was engaged with the Canadian Car and Foundry Company which was working on Government contracts for making war munitions. His first work in this position was in the office preparing drawings for the various factories of the Company, but when it became necessary, under stress of war time, to increase production, he was given charge and looked after the construction of six of these factories.

On the completion of this work, Mr. Fiske entered the employ of the International Paper Company as Chief Designer. He was engaged in this capacity when he was called to serve his country in the World War.

After entering the service, Mr. Fiske was sent to Camp Upton, Long Island, N. Y., but was soon ordered into the Third Officers' Training Camp, from which he was graduated and commissioned a Second Lieutenant of Infantry.

Lieut. Fiske went overseas with the 77th Division and arrived in France on March 27th, 1918. There, he spent three months in the Machine Gun School, at which school he was retained for a time as an Instructor. When the 77th

* Memoir prepared by E. W. Bennison, Assoc. M. Am. Soc. C. E.

Division was ordered to the front, Lieut. Fiske was assigned to Company C, 305th Machine Gun Battalion. While accompanying Lieut., now Capt. Phillips in the redistribution of their machine guns to assist in an attack planned to liberate the "Lost" Battalion of the 308th Infantry, in command of Major Whittelsey, which had been cut off by the enemy in the Argonne Forest, he was hit by a shell and instantly killed, on October 6th, 1918. He was buried on the following Sunday in Min de l'Homme Mort (Communal Cemetery No. 286), the services having been conducted by Chaplain Nye of the 308th Infantry.

In the Government report to his relatives, Lieut. Fiske was rated as having been a fine officer, extremely popular with his men, with a high sense of honor and devotion to duty. His loss is keenly felt by his family and friends.

He is survived by his wife, formerly Miss Linda Pelkington, of Rockville Center, N. Y., and a baby boy who was twelve days old at the time of his father's death, a sister, Mrs. Ralph Stainy, of Hartford, Conn., who is well known in musical circles as Florence Fiske, and his father, Mr. I. L. Fiske, of Lincoln, Nebr. His mother died several years ago.

Lieut. Fiske was a true and fine character, as all who knew him can testify, very reluctant to assert his own virtues, but always praising those of others. It seems to have been the fortune of war that such a capable engineer as he should have been ordered to a machine gun battalion, but it was his nature to take whatever duty was assigned to him without question and to do his best; and although no doubt he would have fitted in the Corps of Engineers and his services therein would have been of more value to the Government, still, perhaps, the following is appropriate:

"I know there are no errors in the great eternal plan
And all things work together for the good of man,
And I know my soul goes onward in its great eternal quest,
I know as I look earthward, whatever is—is best."

Lieut. Fiske was elected a Junior of the American Society of Civil Engineers on January 6th, 1915.

JAMES TAYLOR LANDRETH, Jun. Am. Soc. C. E.*

DIED JUNE 16TH, 1920.

James Taylor Landreth, the son of Olin H. Landreth, M. Am. Soc. C. E., Professor of Engineering in Union College, and Kittie (Taylor) Landreth, was born in Schenectady, N. Y., on October 28th, 1904. He was graduated from the Schenectady High School in 1912, and acquired his technical education at Union College, where he received the degree of B. S. in Civil Engineering in 1916.

Immediately after his graduation, Mr. Landreth entered the employ of the New York Public Service Commission of the First District, with which he continued until July, 1916. At this time, he became associated with the Building and Maintenance Department of the General Electric Company, at Erie, Pa., where he was engaged as Transitman on topographic surveys, Steel Designer on grade-crossing work, and Inspector in charge of the erection of large shop buildings, until early in 1920, when he was transferred to the Power Station at his own request.

Upon the entrance of the United States into the World War, Mr. Landreth secured a leave of absence from his work and successfully passed through an Officers' Training Camp, but later was refused admission to the service on physical grounds, which was a great disappointment to him.

His work was characterized by remarkable thoroughness and attention to detail, and he was doing most commendable work along the line of combustion engineering, heat losses, etc., when the failure of his health forced him to give it up.

His fine qualities of mind and heart, which had been developed by an ideal home life, endeared Mr. Landreth to his associates of all grades, by whom he was greatly respected for his unfailing courtesy and firmness.

His death occurred on June 16th, 1920, at the home of his father in New York City. He is survived by his father, two sisters, and two brothers, and the Engineering Profession has lost one of the most promising of its young men.

Mr. Landreth was a member of the Alpha Delta Phi Fraternity, and was elected a Junior of the American Society of Civil Engineers on November 28th, 1916.

* Memoir prepared by J. W. Hughes, M. Am. Soc. C. E.

KIRBY BALDWIN SLEPPY, Jun. Am. Soc. C. E.*

DIED AUGUST 4TH, 1918.

Kirby Baldwin Sleppy was born in Danville, Pa., on January 29th, 1888. He received his early education in the schools of Northumberland, Pa., and was graduated from the High School of that place in 1905.

Mr. Sleppy began his engineering work as an Apprentice Electrician with the Scranton Electric Construction Company, of Scranton, Pa., with which he served from July, 1905, to July, 1907.

In September, 1907, he entered the Pennsylvania State College, State College, Pa., from which he was graduated in June, 1911, with the degree of B. S. in Civil Engineering. While at college his vacations were spent in railroad and municipal engineering work.

After his graduation Mr. Sleppy was employed until November, 1911, as Assistant Engineer by the Sunbury, Lewisburg and Milton Street Railway Company on the location and construction of three miles of electric railway at Northumberland, Pa.

He then went West and located at Los Angeles, Cal., where from December, 1911, to 1913, he was engaged as Instrumentman with the Bureau of Power and Light, in the Aqueduct Power Department, on the location of a double high-tension transmission line.

From October, 1913, to June, 1914, Mr. Sleppy was in the employ of the Pacific Electric Railway Company, at Los Angeles, Cal., on street railway construction, and from June to November, 1914, he was engaged with the Southern Sierras Power Company.

In February, 1915, he entered the service of Charles H. Lee, M. Am. Soc. C. E., as Assistant Engineer, on civil and hydraulic engineering work. He remained with Mr. Lee until January, 1917, when he was again employed with the Bureau of Power and Light of the City of Los Angeles.

While at Pennsylvania State College, Mr. Sleppy had been a member of the Cadet Corps, and he had also attended the Business Men's Training Camp at Monterey, Cal., in July, 1916. After the United States declared war with Germany, he entered the Reserve Officers' Training Camp at the Presidio, San Francisco, Cal., from which he was graduated, and, later, that at Vancouver Barracks, Washington. On August 15th, 1917, he was commissioned a Captain in the Engineers Officers' Reserve Corps, and appointed Commanding Officer of Company E of the 4th U. S. Engineers, with which regiment he was sent overseas. On August 4th, 1918, Company E entered the Village of Chery Chartreuve for the purpose of clearing the roads for the artillery advance, and while performing this duty Captain Sleppy was instantly killed and six of his men were wounded by a shell thrown by the enemy.

Captain Sleppy was a young man of determination, initiative, and courage, and his work during his short professional career was always of a high order. He was never married, and is survived by his parents and one brother, Lieut. Paul G. Sleppy, who served with the 28th Division in the World War.

Captain Sleppy was elected a Junior of the American Society of Civil Engineers on December 3d, 1912.

* Memoir prepared from information on file at the Headquarters of the Society.

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